Roadway Design Manual

Revised December 2022

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Manual Notice 2022-2

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Manual: *Roadway Design Manual*

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Purpose

The *Roadway Design Manual* has been revised to update Table 3-8 which represents 4R travel lane and shoulder widths for Rural two-lane highways. The Table has been updated to add in separate column categories for a future ADT of 400 - 1500, and for a future ADT of 1500-2000. The values of these newly added columns have been updated accordingly.

Contents

Chapter 3, Section 4

Updated Table 3-8 by updating criteria values for travel lanes and shoulders for rural two-lane highways. Also, updated footnotes in table.

Instructions

This manual, and all revisions, applies to all transportation project development (all modes), whether developed by the department or by other entities. Due to projects that may be further along in development with current criteria, this manual, and all revisions, will be effective for all projects beginning with the September 2023 Letting, and if project schematic or 30% plans have not been approved by November 1st, 2022. The Districts have the options to use these revisions prior to these dates.

Contact

Contact the Roadway Design Section Director of the Design Division at (512) 416-2678 with any questions or comments.

Archives

Past manual notices are available in a PDF archive.

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Chapter 3 — New Location and Reconstruction (4R) Design Criteria

Preface

Non-discrimination

Texas Department of Transportation (TxDOT) policy is to ensure that no person will on the grounds of race, color, national origin, sex, age or disability be excluded from the participation in, be denied the benefits of, or otherwise be subjected to discrimination under any of our programs or activities.

Overview

TxDOT developed the *Roadway Design Manual* to provide guidance in the geometric design of roadway facilities. This manual contains geometric design recommendations and not absolute design requirements.

The *Roadway Design Manual* represents a synthesis of current information and operating practices related to the geometric design of roadway facilities. The presentation of updated design values does not imply that existing facilities are unsafe or mandate improvement projects. Infrastructure projects are by their nature long-lived facilities. While design methodologies are constantly being improved, the implementation of these improvements typically occurs as projects are built, or rebuilt, in future undertakings.

Traditional roadway project development is expanding to include consideration of the impact on non-facility users and the environment. This more complex approach must take into account both the individual project priorities and the relative priorities of the entire roadway system. Therefore, effective design needs to provide not only beneficial design components, but also the most beneficial total roadway system.

While much of the material in the *Roadway Design Manual* can be considered universal in most geometric design applications, many areas are subjective and may need varying degrees of modification to fit local project conditions. The decision to use specific design guidance at a particular location should be based on an engineering study of the location, operational experience, and objective analysis. Thus, while this document provides guidance for the geometric design of highways and streets, it is not a substitute for engineering judgment. Further, while it is the intent that this document provide geometric design guidance, the *Roadway Design Manual* does not represent a legal requirement for roadway design.

Roadway design is a continually evolving process. As additional information becomes available through experience, research, and/or in-service evaluation, this guide will be updated to reflect current state-of-the-practice geometric design guidance for roadway facilities.

Chapter 1 — Design General

Contents:

Section 1 — Overview

Application of Design Guidelines

The criteria contained in the *Roadway Design Manual* (RDM) are applicable to all classes of highways from freeways to two-lane roads. This RDM represents a synthesis of current information and design practices related to highway design.

Since no document can be expected to cover every highway design situation, the guidelines may require modification for local conditions. It is important that significant deviations from the manual be documented and be based on an objective engineering analysis.

It should be noted that roadway design criteria and technology are a rapidly changing field of study. The fact that new design values are presented or updated herein does not imply that existing highway conditions are less safe. Also, continually enhanced design practices do not mandate the need for improvement projects. With significant transportation infrastructure in place, the intention is to use the most current design techniques on future construction projects.

Various environmental impacts can be mitigated or eliminated using appropriate design practices. The result of the application of this manual should result in projects which provide user safety and operational efficiency while taking into account environmental quality. Whereas “desirable” design criteria are generally preferred, the selection of “minimum” design criteria may be warranted to produce a finished project that is more consistent with surrounding terrain and/or settings.

Roadway Design Manual Format

The RDM is formatted to incorporate the following categories of highway construction: resurfacing, restoration, rehabilitation, and reconstruction. The following is a brief description of each chapter:

Chapter 1 describes the requirements of Design Exceptions, Design Waivers, Design Variances, and the Texas Highway Freight Network's Design Deviations. The chapter also summarizes the requirements for schematic layouts and approval, preliminary design, access to the interstate, and maintenance considerations during design.

Chapter 2 presents basic design criteria. Portions of this section will have application to all projects to varying degrees. The chapter discusses traffic characteristics, sight distance, horizontal and vertical alignment, and cross-sectional elements. The dimensions given in this chapter will be referenced for most of the roadway classifications.

Chapter 3 describes new location and reconstruction (4R) project design criteria. These projects usually involve substantial design since they are either new roadways, added capacity projects or almost totally reconstructed roadway sections. The chapter is broken into the following roadway classifications: urban streets, suburban roadways, two-lane highways, multi-lane rural highways, and freeways.

Chapter 4 describes non-freeway rehabilitation (3R) project design criteria. Rehabilitation projects are intended to preserve and extend the service life of the existing roadway and to enhance safety. The chapter presents criteria for improvements and enhancements within the context of acceptable rehabilitation project design.

Chapter 5 describes non-freeway restoration (2R) project design criteria. Restoration projects are intended to restore the pavement structure, riding quality, or other necessary components to their existing cross section configuration. The chapter makes a special note that the addition of through travel lanes is not permitted under a restoration project.

Chapter 6 describes special facility design criteria. Special facilities may include off-system bridge projects, historical roadways or structures, park roads, and bicycle facilities. For these projects, the roadway may have preservation or economic considerations which have equal weight with the user access and mobility characteristics of the roadway, bridge, or other facility.

Chapter 7 describes miscellaneous design elements. These elements may not be a part of all highway projects. Guidance is given concerning longitudinal barriers and roadside safety hardware criteria, fencing, pedestrian separation and ramps, parking, rumble strips, emergency median openings on freeways, and minimum turning designs for trucks and buses. These individual design elements can be selected as needed and incorporated into appropriate project designs.

Chapter 8 describes and provides design guidance on mobility corridors with design speeds of 85 mph to 100 mph. Guidance is given on roadway design criteria, roadside design criteria, ramps, and direct connectors.

Appendix A describes the components of guardrail installations and the methodology for determining appropriate lengths of need.

Appendix B describes the treatment of pavement drop-offs in work zones.

Appendix C provides guidance on the location and design of driveway connections.

Appendix D provides guidance on right-turn slip lanes.

Appendix E provides guidance on alternative intersection designs. An overview, design considerations, pedestrian and bicyclist considerations, and access management are discussed for roundabouts, Diverging Diamond Interchanges (DDI), Median U-Turn Intersections (MUT), Restricted Crossing U-Turn Intersections (RCUT), and Displaced Left Turn Intersections (DLT).

NOTE: Guidance on Preventative Maintenance (PM) projects can be found in TxDOT's Maintenance Management Manual and the Maintenance Operations Manual.

External Reference Documents

It is recommended that the following publications, in their current editions, be available for reference in conjunction with the RDM. These listed publications are produced by entities other than the Texas Department of Transportation.

- *A Policy on Geometric Design of Highway and Streets* (Green Book), American Association of State Highway and Transportation Officials (AASHTO)
- *Roadside Design Guide (AASHTO)*
- *Highway Capacity Manual*, Transportation Research Board (TRB)
- *Highway Safety Manual (AASHTO)*
- *Guide for the Development of Bicycle Facilities (AASHTO)*
- *Guide for the Design of High Occupancy Vehicle Facilities (AASHTO)*
- *Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way* (PROWAG), United States Access Board
- *Guide for the Planning, Design, and Operation of Pedestrian Facilities (AASHTO)*
- *Bikeway Selection Guide (FHWA)*
- *Separated Bike Lane Planning and Design Guide (FHWA)*
- *A Policy on Design Standards-Interstate System (AASHTO)*

AASHTO has established various policies, standards, and guides relating to transportation design practices. These documents are approved references to be used in conjunction with the RDM. However, the instructions given in the RDM will take precedence over AASHTO documents unless specifically noted otherwise.

Section 2 — Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight Network (THFN) Design Deviations

Overview

This subsection discusses the following topics:

- Design Exceptions;
- Design Waivers;
- Design Variances; and
- Texas Highway Freight Network (THFN) Design Deviations.

Design Exceptions

A design exception is required whenever the controlling criteria specified for different categories of construction projects (i.e., 4R, 3R, 2R, Special Facilities, Off-System Bridge Replacement and Rehabilitation projects, Historically Significant Bridge Projects, On-System Park Road Projects, and on-street Bicycle Facilities) are not met. Approval of a design exception rests with the TxDOT District, unless the project is subject to federal oversight or review. A design exception is not required when values exceed the minimum guidelines for the controlling criteria.

As per the *Stewardship and Oversight Agreement on Project Assumption and Program Oversight by and Between Federal Highway Administration, Texas Division and the State of Texas Department of Transportation*, 2015 (FHWA-TxDOT S&O Agreement, 2015), the FHWA has retained approval of design exceptions on the Interstate. Additionally, if a project (Interstate or other) is identified on the FHWA Texas Division's list of "Involved Projects" (TxDIP), and the project's TxDIP plan identifies any design exception responsibility to FHWA, then all proposed design exceptions for that project will be required to be submitted to FHWA. Design exceptions to be submitted to the FHWA, must first be submitted to the Design Division - Project Development Support Section for review; the Design Division - Project Development Support Section will then transmit the design exception to the FHWA for approval. All other project design exceptions are to be approved by the TxDOT District.

The FHWA adopted design requirements in 23 CFR 625, and 49 CFR 37.9 apply to new and reconstruction projects on the NHS. Note, in some instances TxDOT RDM standards are more restrictive than FHWA adopted standards. In cases where FHWA adopted standards are met but TxDOT standards are not met, an FHWA approved design exception is not required and the requests should be processed as an internal TxDOT design exception. For example, an FHWA approved design exception would be required if a minimum 16.0 ft. vertical clearance on an Interstate was not met (as listed in 23 CFR 625.4); since TxDOT has a 16.5 ft. vertical clearance requirement for an Interstate (that does not meet the project criteria for the THFN vertical clearance requirements) then an Interstate vertical clearance of between 16.0 ft. and less than 16.5 ft. would require an internal TxDOT design exception. Additionally, a vertical clearance of less than 16 ft. on the Interstate requires DoD coordination and approval. Vertical clearance coordination is required with the Surface Deployment and Distribution Command Transportation Engineering Agency (SDDCTEA). A form is available through TxDOT Design Division and must be included with the Design Exception approval request to FHWA.

Design exceptions involving the design loading structural capacity or bridge rails must be sent to the Bridge Division for their review and approval.

All design exceptions must be signed by the TxDOT District Engineer and this signature authority cannot be delegated. Design exceptions must be documented on the Form 1002.

The review of design exceptions and recommendations for approval/non-approval may be established individually by each TxDOT District. For example, a four person review committee might be established which would include:

- Director of Transportation Planning and Development;
- Director of Construction;
- Director of Operations/Traffic; and
- Area Engineer (not responsible for project management).

The recommendation of approval/non-approval by a majority of the committee members would constitute a quorum for recommending signature action.

The complete documentation for a roadway design exception should be retained permanently in the District project files. Since the construction plans are sealed, the design exception documentation does not require an engineer's seal.

The following project categories will have controlling criteria that dictate a **design exception**.

New Location and Reconstruction Projects (4R)

Table 1-1 gives the controlling criteria for 4R projects that will require a design exception.

Table 1-1. Controlling Criteria for 4R Projects

Criteria	Reference	Explanation
Design Speed	Table 3-1 Table 3-5 Table 3-7 Table 3-11 Table 3-17	Minimum design speed based on the functional classification.
Lane Width	Table 3-1 Table 3-5 Table 3-8 Table 3-11 Table 3-18	Minimum lane width based on functional classification, type of lane (Mainlane, Speed Change Lane, etc.) and design speed.
Shoulder Width	Table 3-1 Table 3-5 Table 3-8 Table 3-11 Table 3-18	Minimum outside and inside shoulder widths based on functional classification, type of facility and design speed.
Horizontal Curve Radius	Table 2-3 Table 2-4 Table 2-5	Minimum horizontal curve radius based on design speed and maximum superelevation rate.
Superelevation Rate	Table 2-3 Table 2-4 Table 2-5	The amount of cross slope needed on a horizontal curve to help counterbalance the centrifugal force of a vehicle traversing the curve based on the design speed.
Stopping Sight Distance (SSD)[1]	Table 2-1	The distance a vehicle needs to be able to see in order to have room to stop before colliding with something in the roadway. Use k-values to assist calculation.
Maximum Grade	Table 2-9	Maximum gradient of a roadway along the centerline.
Cross Slope	Chapter 2 – Pavement Cross Slope	Transverse slope across a pavement surface.
Vertical Clearance	Table 2-11	Minimum specified height of a bridge or overhead projection above the roadway.
Design Loading Structural Capacity	Bridge Inspection Manual (Ch. 5, Section 3)	The Design Load Structural Capacity-Maximum Load that a bridge is designed to handle.
Bridge Class Culvert Protection	TxDOT Bridge Railing Manual, RDM Ch. 2, Section 8.	Protection of bridge class culvert is not provided for ADT > 1,500.
Bridge Rail	TxDOT Bridge Railing Manual Ch. 4, Section 2.	Bridge rail must comply with mash and meet requirements in Ch. 4, Section 2 of the TxDOT Bridge Railing Manual.

Table 1-1. Controlling Criteria for 4R Projects

Criteria	Reference	Explanation
Note: 1. SSD applies to horizontal alignments, and crest vertical curves for the purposes of a Design Exception. SSD for crest vertical curves is a direct correlation with the K-Value. If the minimum K-Value is satisfied for a crest vertical curve (Fig. 2-6), then the vertical SSD is satisfied under usual conditions.		

Resurfacing, Restoration or Rehabilitation (3R) Projects

Table 1-2 gives the controlling criteria for 3R projects that will require a design exception. For 3R projects, "high-volume" roadways are defined as current ADT of 1,500 and greater.

Table 1-2. Controlling Criteria for 3R Projects

Criteria	Low Volume	High Volume	Reference	Explanation
Deficient Bridge Rails		X	TxDOT **Bridge Railing Manual**	Minimum requirements are met for new bridge rail or existing rail to remain in place.
Design Speed		X	Table 4-1 Table 4-2 Table 4-3 Table 4-4 Table 4-5	Minimum design speed based on the highway class and context.
Horizontal Curve Radius		X	Table 2-3 Table 2-4 Table 2-5	Minimum horizontal curve radius based on design speed and maximum superelevation rate.
Superelevation Rate		X	Table 2-3 Table 2-4 Table 2-5	The amount of cross slope needed on a horizontal curve to help counterbalance the centrifugal force of a vehicle traversing the curve based on the design speed.
Stopping Sight Distance (SSD)[1]		X	Table 2-1	The distance a vehicle needs to be able to see in order to have room to stop before colliding with something in the roadway. Use k-values to assist calculation.
Maximum Grade		X	Table 2-9	Maximum gradient of a roadway along the centerline.
Lane Width	X	X	Table 4-1 Table 4-2 Table 4-3 Table 4-4 Table 4-5	Minimum lane width based on the respective Functional Classification, highway class, and Current Average Daily Traffic.
Shoulder Width	X	X	Table 4-1 Table 4-2 Table 4-3 Table 4-4 Table 4-5	Minimum shoulder width based on the respective Functional Classification, highway class, and Current Average Daily Traffic.

Table 1-2. Controlling Criteria for 3R Projects

Criteria	Low Volume	High Volume	Reference	Explanation
Design Loading Structural Capacity	X	X	Bridge Inspection Manual (Ch. 5, Section 3)	The Design Load Structural Capacity-Maximum Load that a bridge is designed to handle.
Note: 1. SSD only applies to crest vertical curves for the purposes of a Design Exception for 3R high-volume roadways. SSD for crest vertical curves is a direct correlation with the k-value. If the minimum k-value is satisfied for a crest vertical curve (Figure 2-6), then the vertical SSD is satisfied under usual conditions.				

Resurfacing or Restoration Projects (2R)

Design exceptions are required for 2R projects any time the existing geometric or bridge features for the proposed project will be reduced below design standards.

Bicycle Facilities

Table 1-3 gives the controlling criteria for bicycle facility projects that will require a design exception when the minimum requirements given in *AASHTO's Guide for the Development of Bicycle Facilities* cannot be met. See Chapter 6, Section 4, Bicycle Facilities for additional information.

Table 1-3. Controlling Criteria for Bicycle Facilities

Criteria	Reference	Explanation
Bike Lane	Chapter 6, Section 4, Bicycle Facilities	Urban and Rural criteria given.
Shared Lane (Wide Outside Lane)	Chapter 6, Section 4, Bicycle Facilities	Urban and Rural criteria given.
Bridge Deck Clear Space	Chapter 6, Section 4, Bicycle Facilities	Minimum shoulder and offset measured to the toe of barrier provided on the structure.

Special Facilities

Table 1-4 gives the requirements for off-system bridge replacement and rehabilitation projects with current ADT of 400 or less that will require a design exception. If the current ADT is greater than 400, then applicable 4R or 3R criteria applies. See Chapter 6 Section 1, Off-System Bridge Replacement and Rehabilitation Projects.

Table 1-4. Controlling Criteria for Off-System Bridges with < 400 ADT

Criteria	Reference	Explanation
Design Speed	Chapter 6, Section 1, Off-System Bridges	Meet or improve conditions that are typical on the adjacent sections of roadway.
Approach Roadway Width	Chapter 6, Section 1, Off-System Bridges	Minimum approach width, transition, and surfacing.

Table 1-4. Controlling Criteria for Off-System Bridges with < 400 ADT

Criteria	Reference	Explanation
Bridge End Guard Fence	Chapter 6, Section 1, Off-System Bridges	Minimum treatment required.
Horizontal Curve Radius	Chapter 6, Section 1, Off-System Bridges	Meet or improve conditions that are typical on the adjacent sections of roadway.
Superelevation Rate	Chapter 6, Section 1, Off-System Bridges	Meet or improve conditions that are typical on the adjacent sections of roadway.
Stopping Sight Distance (SSD)[1]	Table 2-1	Meet or improve conditions that are typical on the adjacent sections of roadway.
Maximum Grade	Chapter 6, Section 1, Off-System Bridges	Meet or improve conditions that are typical on the adjacent sections of roadway.
Design Loading Structural Capacity	Bridge Inspection Manual (Ch. 5, Section 3)	The Design Load Structural Capacity-Maximum Load that a bridge is designed to handle.

Note:

1. SSD only applies to crest vertical curves for the purposes of a Design Exception for off-system bridge projects. SSD for crest vertical curves is a direct correlation with the K-Value. If the minimum K-Value is satisfied for a crest vertical curve (Table 2-10), then the vertical SSD is satisfied under usual conditions.

Off-System Historically Significant Bridge Projects

Table 1-5 gives the controlling criteria for off-system historically significant bridge projects that will require a design exception. See Chapter 6 Section 1, Off-System Bridge Replacement and Rehabilitation Projects.

Table 1-5. Controlling Criteria for Off-System Historically Significant Bridges

Criteria	ADT	Reference	Explanation
Roadway Width	≤ 400	Chapter 6, Section 1, Off-System Bridges	Meet or improve conditions that are typical on the adjacent sections of roadway.
Load Carrying Capacity (Operating Rating)		Bridge Inspection Manual (Ch. 5, Section 3)	Operating Rating, the maximum permissible live load that can be placed on the bridge. This load rating also includes the same load in multiple lanes. Allowing unlimited usage at the Operating Rating level will reduce the life of the bridge.

Park Road Projects

Design exceptions are not applicable to park road projects that are off the state highway system and designated as Park and Wildlife Roads (PW). Design is based on the criteria and guidance given in the current publication of the *Texas Parks and Wildlife Department Design Standards for Roads and Parking*, or as approved by the Texas Parks and Wildlife Department.

On-system park road projects, designated as Park Roads (PR), must meet the required design criteria for the appropriate roadway classification including design exception or design waiver requirements.

Design Waivers

When the criterion is not met in a non-controlling category, a design exception is not required. However, a departure from the minimum criteria will be handled by design waivers at the District level. Design waivers will be granted as the District authorizes. Complete documentation should be retained permanently in the District project files. On system Park Road projects need to meet the applicable criteria, see Chapter 6, Section 3 for additional information.

The non-controlling criteria for the following project categories will require a design waiver.

New Location and Reconstruction Projects (4R)

Table 1-6 gives the non-controlling criteria for 4R projects that will require a design waiver.

Table 1-6. Non-Controlling Criteria for 4R Projects

Criteria	Reference	Explanation
Curb Parking Lane Width	Table 3-1 Table 3-5	Minimum width required for vehicles to park on the edge of the roadway in urban and suburban areas based on the functional class of the roadway.
Speed Change Lane Width	Table 3-1 Table 3-2 Table 3-5 Table 3-18	Minimum width of acceleration or deceleration lanes for left or right turns, exit or entrance lanes, or a climbing lane based on the highway class.
Length of Speed Change Lanes	Table 3-3 Table 3-4 Table 3-12 Table 3-13 Table 3-18 Table 3-23 Figure 3-44	Minimum length of acceleration or deceleration lanes for left or right turns, exit or entrance lanes, or a climbing lane based on the highway class. Minimum Spacing between ramps as shown in Figure 3-44, or acceptable LOS not met.
Curb Offset	Table 3-1 Table 3-5	Minimum distance between the edge of the travel lane and the face of the curb.
Bridge Width	TxDOT Bridge Project Development Manual	Minimum overall bridge width.
Median Opening Width	Table 3-1 Table 3-2 Table 3-5 Table 3-11	Minimum width of separation between dual roadways which separates the traffic flow in the opposite direction.
Clear Zone	Table 2-12	The unobstructed, traversable area provided beyond the edge of the through traveled way for the recovery of errant vehicles.

Table 1-6. Non-Controlling Criteria for 4R Projects

Criteria	Reference	Explanation
Lateral Offset to Obstructions	Chapter 2, Lateral Offset to Obstructions	Minimum distance from the edge of the traveled way, beyond which a roadside object will not be perceived as an obstacle and result in a motorist's reducing speed or changing vehicle position on the roadway.
Railroad Overpass Geometrics	TxDOT Bridge Project Development Manual	Minimum vertical and horizontal clearances.
Sag Vertical Curve Length[1]	Figure 2-1 Figure 2-7	Length of sag vertical curve.
Superelevation (Non-rate elements)	Chapter 2, Section 5 Horizontal Alignment	Non-rate elements include length and location of superelevation transitions.
Guardrail Length (unless for access accommodation)	Appendix A, Longitudinal Barriers	The Length of Need (LON) of guardrail required to protect traffic adjacent to guardrail, as well as apposing traffic.
Shared Use Path	Chapter 6, Section 4, Bicycle Facilities	Bikeways that are physically separated from vehicular traffic and allows pedestrians and other non-motorized users.
Bridge Class Culvert Protection	TxDOT Bridge Railing Manual, and RDM Ch. 2, Section 8	Protection of bridge class culvert is not provided for ADT ≤ 1500.

Note:

1. Sag Vertical Curve Length applies to sag vertical curves for the purposes of a 4R Design Waiver. Sag Vertical Curve Length for vertical alignment is a direct correlation with the k-value. If the minimum k-value is satisfied for a sag vertical curve (Figure 2-7), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings see Figure 2-1. See Chapter 2, Section 6 for additional guidance on comfort criteria.

Resurfacing, Restoration or Rehabilitation (3R) Projects

Table 1-7 gives the non-controlling criteria for 3R projects that will require a design waiver. For 3R projects, "low-volume" roadways are defined as a current ADT of less than 1500.

Table 1-7. Non-Controlling Criteria for 3R Projects

Criteria	Low Volume	High Volume	Reference	Explanation
Design Speed	X		Table 4-1 Table 4-2 Table 4-3 Table 4-4 Table 4-5	Minimum design speed based on the functional classification.
Horizontal Curve Radius	X		Table 2-3 Table 2-4 Table 2-5	Minimum horizontal curve radius based on design speed and maximum superelevation rate.
Superelevation Rate	X		Table 2-3 Table 2-4 Table 2-5	The amount of cross slope needed on a horizontal curve to help counterbalance the centrifugal force of a vehicle traversing the curve based on the design speed.

Table 1-7. Non-Controlling Criteria for 3R Projects

Criteria	Low Volume	High Volume	Reference	Explanation
Stopping Sight Distance (SSD)[1]	X		Table 2-1	The distance a vehicle needs to be able to see in order to have room to stop before colliding with something in the roadway. Use k-values to assist calculation.
Maximum Grade	X		Table 2-9	Maximum gradient of a roadway along the centerline.
Superelevation (Non-rate elements)	X	X	Chapter 2, Section 5 Horizontal Alignment	Non-rate elements include length and location of superelevation transitions
Bridge Width	X	X	TxDOT Bridge Project Development Manual	Minimum overall bridge width.
Deficient Bridge Rails	X		TxDOT Bridge Railing Manual	Minimum requirements are met for new bridge rail or existing rail to remain in place.
Clear Zone	X	X	Table 4-1 Table 4-2 Table 4-3 Table 4-4 Table 4-5	The unobstructed, traversable area provided beyond the edge of the through traveled way for the recovery of errant vehicles.
Turn Lane Width	X	X	Table 4-1 Table 4-2 Table 4-3	Minimum width of speed change lanes.
Length of Speed Change Lanes	X	X	Table 3-3 Table 3-4 Table 3-12 Table 3-13 Figure 3-44	Minimum length of acceleration or deceleration lanes for left or right turns, exit or entrance lanes, or a climbing lane based on the highway class. Minimum Spacing between ramps as shown in Figure 3-44, or acceptable LOS not met.
Curbside Parking Lane Width (Parallel Parking)	X	X	Table 4-3	Minimum width of parallel parking lanes based on the functional class of the roadway.
Guardrail Length (unless for access accommodation)	X	X	Appendix A, Longitudinal Barriers	The Length of Need (LON) of guardrail required to protect traffic adjacent to guardrail, as well as apposing traffic.
Sag Vertical Curve Length [2]	X	X	Figure 2-1 Figure 2-7	Length of sag vertical curve.
Shared Use Path	X	X	Chapter 6, Section 4, Bicycle Facilities	Bikeways that are physically separated from vehicular traffic and allows pedestrians and other non-motorized users.

Table 1-7. Non-Controlling Criteria for 3R Projects

Criteria	Low Volume	High Volume	Reference	Explanation
Notes: 1. SSD only applies to crest vertical curves for the purposes of a Design Exception for 3R high-volume roadways. SSD for crest vertical curves is a direct correlation with the k-value. If the minimum k-value is satisfied for a crest vertical curve (Figure 2-6), then the vertical SSD is satisfied under usual conditions. 2. Sag Vertical Curve Length applies to sag vertical curves for the purposes of a 3R Design Waiver. Sag Vertical Curve Length for vertical alignment is a direct correlation with the K-Value. If the minimum K-Value is satisfied for a sag vertical curve (Figure 2-7), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings, see Figure 2-1. See Chapter 2, Section 6 for additional guidance on comfort criteria.				

Resurfacing or Restoration Projects (2R)

Design waivers are not applicable to 2R projects.

Bicycle Facilities

Table 1-8 gives the non-controlling criteria for bicycle facility projects that will require a design waiver when the minimum requirements given in AASHTO's *Guide for the Development of Bicycle Facilities* cannot be met. See Chapter 6, Section 4, Bicycle Facilities.

Table 1-8. Non-Controlling Criteria for Bicycle Facilities

Criteria	Reference	Explanation
Shared Use Path (Independent alignment or Side Path)	Chapter 6, Section 4, Bicycle Facilities	Urban and Rural criteria given.
Separated Bike Lane/Buffered Bike Lane	Chapter 6, Section 4, Bicycle Facilities	Urban and Rural criteria given.
Bike Accessible Shoulder Width	Chapter 6, Section 4, Bicycle Facilities	Urban and Rural criteria given.

Special Facilities

Table 1-9 gives the non-controlling criteria for off-system bridge replacement and rehabilitation projects with current ADT of 400 or less that will require a design waiver.

Table 1-9. Non-Controlling Criteria for Off-System Bridges with < 400 ADT

Criteria	Reference	Explanation
Superelevation (Non-rate elements)	Chapter 2, Section 5, Horizontal Alignment	Superelevation (Non-rate elements)
Sag Vertical Curve Length[1]	Figure 2-1 Figure 2-7	Length of sag vertical curve.
Bridge Width	Chapter 6, Section 1, Off-System Bridges	Minimum Structure Width, see Chapter 6 Section 1 for criteria.

Table 1-9. Non-Controlling Criteria for Off-System Bridges with < 400 ADT

Criteria	Reference	Explanation
Note: 1. Sag Vertical Curve Length applies to sag vertical curves for the purposes of a 3R Design Waiver. Sag Vertical Curve Length for vertical alignment is a direct correlation with the k-value. If the minimum k-value is satisfied for a sag vertical curve (Figure 2-7), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings, see Figure 2-1. See Chapter 2, Section 6 for additional guidance on comfort criteria.		

Off-System Historically Significant Bridge Projects

Design waivers are not applicable to off-system historically significant bridge projects.

Park Road Projects

Design waivers are not applicable to or off-system park road projects.

Design Variances

TxDOT designers must use PROWAG to achieve accessible design requirements in the public right-of-way. TAS and DOJ ADA Standards (2010) must be used for design and construction of buildings and sites such as TxDOT buildings, Safety Rest Areas, etc. A request for a design variance for any deviations from the applicable PROWAG, or if applicable, TAS requirements must be submitted to the TDLR for approval. Design variances should be sent to the Design Division – Landscape Architecture Section for review prior to being submitted to TDLR for approval. Refer to Chapter 7, Section 3, Pedestrian Facilities for additional discussion. This documentation should be kept in the project files.

A Design Variance form, process flowchart, and other information can be found on the Design Division's website.

Texas Highway Freight Network (THFN) Design Deviations

A Texas Highway Freight Network (THFN) design deviation is required for projects that do not meet specified Bridge Vertical Clearance Requirements; see Chapter 3, Section 8 for applicable project criteria, and Table 2-11 for vertical clearance values. The Deviation request is exclusive of (and in addition to) the design exception that may be needed for Vertical Clearances not meeting the specified requirements in Chapter 3 of the respective roadway facility. The THFN Deviation requests must be submitted to the Design Division – Project Development Support Section, and will be reviewed by an assigned Design Deviation Committee.

A Design Deviation form, process flowchart, and other information can be found on the Design Division’s website.

Section 3 — Schematic Development

Overview

This subsection discusses the following topics:

- Schematic Development Process;
- Schematic Layout; and
- Schematic Approval.

Schematic Development Process

See the Project Development Process Manual for a comprehensive discussion of the schematic development process.

Schematic Layout

Schematic layouts should include the basic information necessary for a proper review and evaluation of the proposed improvement. The following should be included at a minimum and the adequacy verified with each submission of a schematic:

- A Design Summary Report (DSR) appropriate for the stage of the project development;
- General project information, including TxDOT District, County, Control-Section-Job (CSJ), roadway name, project limits, project length, stationing equations, and project location map;
- Design speed, functional classification, and existing and projected ADT volumes for each roadway, including mainlanes, frontage roads, ramps, direct connectors, and cross roads;
- TxDOT logo and copyright, the Engineer's firm name (if applicable), the Engineer's certification per the Texas Board of Professional Engineers rules, the statement "Not a Bidding Document", the percent complete, and the date of latest revision;
- North arrow, legend, drawing scales, and labeling for the beginning and ending of the project;
- Traffic diagram(s) showing existing and projected ADT volumes of mainlanes, ramps, frontage roads, turnarounds, and cross roads, and the breakdown of those volumes for turning movements, as applicable;
- Proposed horizontal and vertical alignments for proposed mainlanes, ramps, frontage roads, ancillary roads, and cross roads, including labels for all curve data, elevations, grades, and minimum vertical clearances over highways superelevation rates, locations, and transition lengths;

- Existing and proposed bicycle and pedestrian accommodations, including curb ramp locations and roadway crossing locations;
- Existing and proposed horizontal alignments for railroads;
- Existing roadways in plan-view and existing ground profiles and elevations along each proposed alignment;
- Existing vertical alignments for cross roads to remain in place at proposed interchanges or grade separations;
- Pavement marking and arrows indicating the number of lanes, direction of travel, and lengths of lane/shoulder tapers and weaving areas;
- Aerial imagery or an orthophotograph in the plan view, if available;
- Locations and widths of medians and median openings;
- Lengths and geometrics of speed change lanes, turn lanes, auxiliary lanes, and storage bays;
- Portions of existing roadways and structures to be removed or closed;
- Existing and proposed right of way (ROW) limits, property lines, and driveways;
- Existing and proposed control-of-access (COA) lines;
- Relevant affected utilities;
- Existing and proposed bridges and bridge class culverts, including the beginning and ending stations of each proposed structure;
- Locations of retaining walls and/or noise walls;
- Existing and proposed typical sections for mainlanes, ramps, frontage roads, ancillary roads, and cross roads, including the following elements:
  - Proposed and future lane and shoulder widths
  - Existing and proposed roadway centerline
  - Direction of travel and turn movements allowed (right/left turn, option lane, etc.)
  - Curb offsets
  - Median widths
  - Clear zone widths
  - Existing and proposed ROW width
  - Pedestrian and bicycle accommodations
  - Sidewalk and bicycle accommodation widths and cross slopes, including buffer widths
  - Slope rates of front and back slopes
  - Roadside barrier nominal widths and offsets
  - Roadway cross-slopes

- If the proposed design is interim, include typical sections for the ultimate design with the above parameters indicated;
- Locations of wildlife crossing structures;
- Vertical location of cross-drainage structures in the profile view;
- Rivers, major stream crossings, FEMA floodplain boundaries, and the anticipated water surface for the required design event;
- Locations of traffic signals;
- Locations, structure type, and sign composition of existing and proposed freeway signing, including mile markers; and
- A written explanation or prepared exhibits of the sequence and methods of staged construction, including the treatment of crossovers and ramps.

Each schematic should be able to stand alone, even if part of a larger project. The proposed roadway should tie into the existing roadway as it will be at the time of construction. Any adjacent projects (concurrent or future) should be shown and labeled as “By Others”, “Future” or similar.

For freeway added capacity and interchange/ramp modification projects, and major highway and intersection reconstruction projects, traffic operational and safety analyses are recommended. The traffic operational analysis is based on the Highway Capacity Manual (HCM) or microsimulation tools. The safety analyses should include a historical crash data analysis for at least four years and a predictive crash analysis as detailed in the Highway Safety Manual (HSM). For guidance on the scope of operational and safety analyses and applicable tools, contact the Design Division.

A comprehensive checklist for schematics, covering the elements above as well as other documentation requirements and considerations, can be found on the Design Division Project Development Support Section website.

Schematic Approval

All schematics are to be submitted to the Design Division for review. If a project is on the FHWA’s Texas Division list of Involved Projects (TxDIP) and the project’s TxDIP *Risk Analysis Stewardship & Oversight Plan* (TxDIP Plan) identifies that the project’s schematic is to be reviewed by the FHWA, the Design Division will submit the schematic to the FHWA for review. FHWA does not approve schematics unless specified in the TxDIP S&O Plan, FHWA will approve IAJRs and Design Exceptions, which are supported by a schematic, as appropriate (see previous section on Design Exceptions for additional information). For all other schematics, approvals will be at the District level.

Section 4 — Access to the Interstate System

According to United States Code (USC) - Title 23 Section 111, proposed new or revised access points to the Interstate system require review and approval by the FHWA. The FHWA's decision to approve a request is dependent on the proposal satisfying and documenting the two following requirements:

- Operational and Safety Analysis; and
- Access and Design.

According to federal regulations, the application of these requirements is as follows:

- These requirements are applicable to new or revised access points to existing Interstate facilities regardless of the funding of the original construction or the funding for the new access points. This applicability includes routes incorporated into the Interstate system under the provisions of 23 U.S.C. 103(c)(4)(A) or other legislation.
- Routes approved as a future part of the Interstate system under 23 U.S.C. 103(c)(4)(B) represent a special case because they are not yet a part of the Interstate system. Any proposed new or significant changes in access beyond those covered in the agreement, regardless of funding must be approved by FHWA.
- Toll roads that are incorporated into the interstate system do not require FHWA coordination and approval, except for segments where federal funds have been expended or these funds will be used for roadway improvements, or where the toll road section has been added to the Interstate system under the provisions of 23 U.S.C. 103(c)(4)(A). The term “segment” is defined as the project limits described in the Federal-aid project agreement.
- Each break in the control of access to the Interstate system right-of-way is considered to be an access point. Each entrance or exit point, including “locked gate” access, is considered to be an access point. For example, a diamond interchange configuration has four access points.
- Ramps providing access to rest areas, information centers, and weigh stations within the Interstate controlled access are not considered access points for applying these requirements. These facilities must be accessible to vehicles only to and from the Interstate system. Access to or from these facilities and local roads and adjoining property is prohibited. The only allowed exception is for access to adjacent publicly owned conservation and recreation areas, if access to these areas is available only through the rest area, as allowed under 23 CFR 752.5(d).
- Generally, any change in the design of an existing access point is considered a change to the interchange configuration even though the number of actual points of access may not change. For example, replacing one of the direct ramps of a diamond interchange with a loop or changing a cloverleaf interchange into a fully directional interchange would be considered revised access for the purpose of applying these requirements.

- All requests for new or revised access points on completed Interstate highways must closely adhere to the planning and environmental review processes as required in 23 CFR 450 and 771.
- All requests for new or revised access points on completed Interstate highways must be closely coordinated with the planning and environmental processes. The FHWA approval constitutes a federal action, and as such, requires that the transportation planning, conformity, congestion management process, and the National Environmental Policy Act (NEPA) procedures be followed and their requirements satisfied. The final FHWA approval of requests for new or revised access cannot precede the completion of these processes or necessary actions.
- 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 must be reevaluated if the project has not progressed to construction within three calendar 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 justification report 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).

In concurrence with FHWA, TxDOT's policy is to add the documentation of the six points (August 2009 FHWA policy) addressing the consideration of social, economic, and environmental impacts and planning considerations (required for NEPA documentation purposes) to the documentation of the May 2017 FHWA two-point policy (focus on safety, operational and engineering issues). Refer to the TxDOT *Interstate Access Justification Standard Operating Procedures* (IAJR SOP) for further guidance on the development of Interstate Access Justification Reports and submittal process through the Design Division. A copy of the current TxDOT IAJR policy is available on Design Division webpage.

Section 5 — Preliminary Design

As outlined in Chapters 2 through 5 of the *Project Development Process Manual*, developing a project for construction letting is a multi-stage process involving many disciplines. The preliminary design process should clearly establish the design criteria, adhering to the *Roadway Design Manual* and other applicable guidelines and policies. Preliminary design processes that affect and influence decisions include the following:

- Survey and mapping [Ground model and topographic/orthographic maps]
- Design Concept Conference (DCC) [Design Summary Report (DSR) and Form 1002]
- Schematic development [Schematic, Interstate Access Justification Report (IAJR), and Frontage Road Briefing Document (FRBD)]
- Traffic operational analyses [Traffic Impact Analysis (TIA), Traffic Analysis Report, etc.]
- Safety review assessment or safety analysis [Safety Index or Crash Models]
- Design Exception/Waiver/Variation/Deviation review [E/W/V/D Approval Documentation]
- ROW determination [Schematic]
- Existing and anticipated land use and development determination
- Constructability review
- Third party agreements [Advance Funding Agreement (AFA), Railroad Agreement, ROW easement, etc.]
- Freeway signing layouts [Schematic]
- Access management / driveway permitting
- Bicycle and pedestrian accommodations
- External agency requirements such as Coast Guard (USCG), emergency management (FEMA), Army Corps of Engineers (USACE), railroads, aviation (FAA), ADA compliance (TDLR), and historical (THC)
- Environmental controls/constraints and clearance [Environmental Document: CE, EA, EIS, etc.]
- Utility impacts / accommodation [Subsurface Utility Exploration (SUE) documentation]
- Geotechnical investigation [Geotech Report]
- Pavement design [Pavement Design Report]
- Value Engineering (VE) Study [VE Report and Executive Decision Summary (Form 2502)]
- PS&E development timeline
- Bridge layouts [Preliminary Bridge Layout Review (PBLR) approval]

- Retaining wall and sound wall layouts
- Hydrologic and hydraulic design [Hydraulic Analysis Report]
- Bridge scour evaluation [Scour Summary Sheet]
- Illumination design
- Traffic control devices [Traffic signal authorization]
- Intelligent Transportation Systems (ITS)
- Landscape and aesthetic design
- Construction timeline determination [Construction Timeline]

NOTE: Brackets [ ] indicate resultant documentation.

The following manuals should be referenced for required submittals and submittal processes:

Surveying:	See *TxDOT Survey Manual*
Design Exceptions, Waivers, Variances, and Deviations:	See *Roadway Design Manual (RDM)* and *Project Development Process Manual (PDP)*
Schematics and IAJRs:	See *RDM* and *PDP Manuals*
ROW Establishment:	See *ROW Acquisition Manual*
Driveways:	See *Access Management Manual* and *RDM*
PS&E Documentation:	See *PS&E Preparation Manual* and *PDP Manual*
Utility Documentation:	See *ROW Utilities Manual*
Railroad Coordination (See Note):	See *Rail-Highway Operations Manual*
Pavement Design:	See *Pavement Manual*
Bridge & Bridge Class Structures:	See *Bridge Project Development Manual*
Retaining Wall Layouts:	See *Geotechnical Manual*
Hydraulic Analysis:	See *Hydraulic Design Manual*
Traffic Control Devices	See *Texas Manual on Uniform Traffic Control Devices*
Traffic Signals:	See *Traffic Signals Manual*
Speed Zones:	See *Procedures for Establishing Speed Zones Manual*
Illumination:	See *Highway Illumination Manual*
Landscape and Aesthetic Design:	See *Landscape and Aesthetics Design Manual*
Local Government Projects:	See *Local Government Projects Policy Manual*
Design Build Projects:	See *Design-Build Contract Administration Manual*

NOTE: Work on or within 500-ft of railroad right-of-way, as measured longitudinally along the project's roadway or laterally along any adjacent road, should be evaluated for impacts. Railroad coordination and letter of agreement are required for any work on or within 50-ft of railroad right-of-way.

Section 6 — Maintenance Considerations in Design

The future maintenance of a facility cannot be overemphasized in project design. Projects which are difficult or costly to maintain, or those which require frequent maintenance activities, are considered to be inadequately designed. Reduced or low maintenance designs with limited worker exposure should be the ultimate goal.

In addition to a maintenance perspective review during project design, it may be appropriate to develop a specific list of design practices to address maintenance needs in a particular area. Such a list might include the following items:

- Acquire drainage easements when necessary to grade outfalls and thus provide adequate drainage. Avoid instances where adjacent property elevation is well above the drainage outfall as this may form a dam at the outfall to the structure.
- Where practical, try to match the drainage structure to the natural grade of the drainage channel, and then profile the roadway over the structure. This practice may reduce siltation in the structure and erosion at the outfall.
- Avoid placing signs in the ditch. Such placement may impede drainage (making mowing more difficult) and result in erosion or siltation around the sign support.
- Where practical, riprap mow strips around sign supports may minimize the need for herbicidal treatment.
- Address access control discrepancies (perhaps due to changes in property ownership) at ramp gores during design.
- Avoid the use of roadside barriers if the fixed object (culvert, large sign, steep slope, etc.) can be appropriately relocated or eliminated. The barrier itself represents a fixed object and should only be used where alternatives are impractical.
- When designing grade separations, consider extending riprap on the header banks of the overpasses all the way to the cross road pavement. This eliminates the need to mow or maintain a small strip of soil under the structure.
- Consider the provision of a narrow mow strip at the bottom or top of retaining walls to simplify mowing operations along the wall. Riprap considerations may also be appropriate in other locations (sign structures, narrow borders, cable barrier, etc.).
- Generally, designs should reduce the amount of hand trimming that would be required and eliminate the places that are relatively difficult for mowers to access.
- Provide access to areas requiring maintenance (mowing, bridge inspection, etc.). For twin bridge structures, provide sufficient distance (typically 10 ft minimum between) to facilitate access for inspections that may require aerial-vehicles.

- To the extent practical, utilization of desirable design criteria recommended herein regarding maximum roadway side slope ratios and ditch profile grades will reduce maintenance and make required maintenance operation easier to accomplish.

Chapter 2 — Basic Design Criteria

Contents:

Section 1 — Overview

Introduction

Designs for highway and bridge projects are based on established design controls and criteria for the various elements of the project such as width of roadway, side slopes, horizontal and vertical alignment, drainage considerations and intersecting roads. This chapter provides descriptions of the traffic characteristics to be considered in selection of design controls and criteria.

Section 2 — Functional Classifications

Overview

The first step in the design process is to define the function that the facility is to serve. The two major considerations in functionally classifying a roadway are access and mobility. Access and mobility are inversely related - that is, as access is increased, mobility is decreased. Roadways are functionally classified first and then contextually classified as either urban, suburban, or rural. The hierarchy of the functional highway system within either the urban, suburban, or rural area consists of the following:

- **Freeways** – controlled access facilities (Interstate, Freeways, and Expressways);
- **Principal arterial** – main movement (high mobility, limited access);
- **Minor arterial** – interconnects principal arterials (moderate mobility, limited access);
- **Collectors** – connects local roads to arterials (moderate mobility, moderate access); and
- **Local roads and streets** – permits access to abutting land (high access, limited mobility).

The functional classification of roadways can be viewed on the Statewide Planning Map that is maintained by TxDOT's Transportation Planning and Programming (TPP) Division. The Statewide Planning Map, however, makes the appropriate distinction that freeways are in fact principal arterials. For purposes of design, freeways have unique geometric criteria that demand a separate design designation apart from other arterials.

Section 3 — Traffic Characteristics

Overview

Information on traffic characteristics is vital in selecting the appropriate geometric features of a roadway. Necessary traffic data includes:

- Traffic Volume;
- Speed;
- Terrain;
- Safety; and
- Additional Considerations.

Traffic Volume

Traffic volume is an important basis for determining what improvements, if any, are required on a highway or street facility. Traffic volumes may be expressed in terms of average daily traffic (ADT) or design hourly volumes (DHV). These volumes may be used to calculate the service flow rate, which is typically used for evaluations of geometric design alternatives and safety analysis.

Average Daily Traffic

ADT represents the total traffic for a year divided by 365, or the average traffic volume per day. Due to seasonal, weekly, daily, or hourly variations, ADT is generally undesirable as a basis for design, particularly for high-volume facilities. ADT should only be used as a design basis for low and moderate volume facilities, where more than two lanes are not justified.

Traffic Forecast

Project level daily travel forecasts are developed and approved by the Transportation Planning and Programming Division (TPP). Generally, projected traffic volume is expressed as ADT with peak (K) and directional (D) factors provided. For high-volume facilities, a tabulation showing traffic converted to DHV or directional design hourly volume (DDHV) will be provided by TPP if specifically requested.

There are generally three approaches to developing daily traffic forecasts:

- **Metropolitan Planning Organization (MPO) Travel Demand Model:** MPO's travel demand model is used to estimate traffic on the project for opening and design year.
- **Pivot/Trend Line/Growth Method:** A growth rate is developed using the historical average annual daily traffic data for 20 years and projected for the next 20 years (pivot year). Growth

factors are used to convert existing year traffic to opening year traffic and opening year traffic to design year traffic.

- **Hybrid Approach:** This approach uses a combination of the first and second methods. MPO's travel demand model is used for developing traffic projections and adjustments are made using growth factors developed by historical or trend line analysis.

There are three options for obtaining approval of daily travel forecasts:

- **Option A:** TPP develops the traffic forecasts, signs and seals the data, and provides the data to the Districts and project consultants.
- **Option B:** Districts and project consultants are responsible for developing the traffic forecasts. TPP reviews and approves the methodology prior to development, reviews and approves the traffic forecasts, and signs and seals the data.
- **Option C:** Districts and project consultants are responsible for developing traffic forecasts. Districts are also responsible for developing the methodology, and developing, reviewing, approving, signing and sealing the traffic forecasts.

Design Hourly Volume

The peak DHV is usually the 30th highest hourly volume for the design year, which is commonly 20 years from the time of expected construction completion. For situations involving high seasonal fluctuations in ADT, some adjustment of DHV may be appropriate.

Computation of DHV and DDHV

For one-way facilities, the ADT is the total traffic volume. For two-lane, two-way, rural highways without major intersections (i.e. intersections where two arterial roads cross) or where additional lanes are not anticipated for the foreseeable future, the volumes are relatively balanced in both directions. Therefore, the ADT_{NDIR} is the total traffic in both directions of travel (i.e. non-directional).

The percent of ADT occurring in the design hour (K) may be used to convert non-directional ADT to DHV as follows:

$DHV = (ADT_{NDIR})(K)$

For urban and metropolitan areas, traffic volumes often show significantly different directional distribution, especially at the interchanges/intersections during AM and PM peak durations. In some cases, significant traffic occurs during mid-day and weekends. Traffic volumes for peak hour or peak period are vital in developing existing and future design transportation needs. Review of 24-hour traffic volume profiles at key locations will determine the peak hour/period. Estimating future traffic volumes in AM and PM peak periods can be a complex process. Refer to Design Division's (Traffic Simulation and Safety Analysis Section) webpage for additional information and guidance.

On two-way facilities with more than two lanes (or on two-lane, two-way facilities where major intersections are encountered or where additional lanes are to be provided later), knowledge of the directional distribution of traffic during the design hour, Directional Design Hourly Volume (DDHV), is essential for design. DHV and DDHV may be determined by the application of conversion factors to ADT.

The K-factor and the percent of directional distribution (D) are both considered in converting non-directional ADT to DDHV, as follows:

$DDHV = (ADT_{NDIR})(K)(D)$

The percentage of ADT occurring in the design hour (K) and the design volume that is in the predominant direction of travel (D) are both considered, and doubled, in converting ADT to DHV as follows:

$DHV = (ADT)(K)(D)(2)$

Directional Distribution (D)

Traffic tends to be more equally divided by direction near the center of an urban area or on loop facilities. For other facilities, the directional distribution is frequently close to 60 to 70 percent.

K Factors (K)

K is the percentage of ADT representing the 30th highest hourly volume in the design year. For typical main rural highways, K-factors generally range from 12 to 18 percent. For urban facilities, K factors are typically lower, ranging from 8 to 12 percent.

Service Flow Rate (SF)

A facility should be designed to provide sufficient capacity to accommodate the design traffic volumes (ADT, DHV, DDHV). The necessary capacity of a roadway is initially based on a set of "ideal conditions." These conditions are then adjusted for the "actual conditions" that are predicted to exist on the roadway section. This adjusted capacity is referred to as the service flow rate (SF) and is defined as a measure of the maximum flow rate under prevailing conditions.

Adjusting for prevailing conditions involves adjusting for variations in the following factors:

- Lane Width;
- Lateral Clearances;
- Free-flow Speed;
- Terrain; and
- Distribution of Vehicle Type.

Service flow rate is the traffic parameter most commonly used in capacity and level-of-service (LOS) evaluations. Knowledge of highway capacity and LOS is essential to properly fit a planned highway or street to the requirements of traffic demand. Both capacity and LOS should be evaluated in the following analyses:

- Selecting geometric design for an intersection;
- Determining the appropriate type of facility and number of lanes warranted;
- Performing ramp merge/diverge analysis; and
- Performing weaving analysis and subsequent determination of weaving section lengths.

All roadway design should reflect proper consideration of capacity and level of service procedures as detailed in the Transportation Research Board's (TRB) *Highway Capacity Manual*.

Speed

Speed is one of the most important factors considered by travelers in selecting alternative routes or transportation modes. In addition to capabilities of the drivers and their vehicles, the speed of vehicles on a road depends upon five general conditions: the physical characteristics of the roadway, the amount of roadside interference, the weather, the presence of other vehicles, and speed limitations established by law or by traffic control devices. Although any one of these factors may govern travel speed, the actual travel speed on a facility usually reflects a combination of factors.

The objective in design of any engineered facility used by the public is to satisfy the public's demand for service in an economical, efficient, and safe manner with low crash frequency and severity. The facility should accommodate nearly all demands with reasonable adequacy and should only fail under severe or extreme traffic demands. Because only a small percentage of drivers travel at extremely high speed, it is not practical to design facilities for these speeds. On the other hand, the speed chosen for design should not be based on the speeds drivers use under unfavorable conditions (such as inclement weather), because the roadway would then be inefficient, might result in additional crashes under favorable conditions, and would not satisfy reasonable public expectations for the facility.

There are important differences between design criteria applicable to low-speed and high-speed designs. For design purposes, the following definitions apply:

- Low-speed is 45 mph and below, and
- High-speed is 50 mph and above.

Design Speed

Design speed is a selected speed used to determine the various geometric design features of the roadway. The selected design speed should be logical with respect to the anticipated operating speed, topography, adjacent land use, modal mix, and functional classification of the roadway. In selection of design speed, every effort should be made to attain a desired combination of safety, mobility, and efficiency within the constraints of environmental quality, economics, aesthetics, and social or political impacts.

The selected design speed should generally be greater than or equal to the anticipated operating speed of the roadway. A roadway of higher functional classification may justify a higher design speed than a lower classified facility in similar terrain. A low design speed should not be selected where the topography is such that drivers are likely to travel at high speeds. Factors to consider when choosing a design speed include the expectations of drivers which are closely related to traffic volume conditions, potential traffic conflicts, and terrain features.

Appropriate design speed values for various highway classes are presented in subsequent sections.

Operating Speed

Operating speed is the speed at which drivers are observed operating their vehicles during free-flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or geometric feature. The following geometric design and traffic demand features may have direct impacts on operating speed: horizontal curve radius, grade, access density, median treatments, on-street parking, signal density, vehicular traffic volume, lane widths, sight distance, and pedestrian and bicycle activity.

Posted Speed

Posted speed refers to the maximum speed limit posted on a section of highway. *The Procedures for Establishing Speed Zones Manual* states that the posted speed should be based primarily upon the 85th percentile speed when adequate speed samples can be secured. Speed zoning guidelines permit consideration of other factors such as roadside development, road and shoulder surface characteristics, public input, and pedestrian and bicycle activity.

Running Speed

The speed at which an individual vehicle travels over a highway section is known as its running speed. Running speed is calculated by dividing the length of the highway section by the time for a typical vehicle to travel through the section. For extended sections of roadway that include multiple roadway types, the average running speed is the most appropriate measure for evaluating level of service and road user costs. The average running speed is the sum of the distances traveled by vehicles on a highway section during a specified period of time divided by the sum of the travel times.

The average running speed on a given roadway varies during the day, depending primarily on the traffic volume. Therefore, when reference is made to a running speed, clearly state whether this speed represents peak hours, off-peak hours, or an average for the day. It is most appropriate to use peak and off-peak running speeds in design and operation. Average running speeds for an entire day should be reserved for economic analyses. The effect of traffic volume on average running speed can be determined using the procedures of TRB's *Highway Capacity Manual*.

Terrain

Terrain classifications pertain to the general character of a specific route corridor. The terrain classification determines the maximum allowable grades in relation to design speed. Selection of classification should be chosen on the surrounding terrain of the corridor and not on the roadway profile grade and embankment slopes.

Level or rolling are the two types of terrain often presented when choosing appropriate design criteria since these are the predominate terrains in Texas. Some areas of the El Paso District and some areas of other western Districts may be considered mountainous. When mountainous conditions are encountered, refer to AASHTO's *A Policy on Geometric Design for Highways and Streets* for appropriate design criteria and design considerations.

Level Terrain

Level terrain is where highway sight distances, as governed by both horizontal and vertical restrictions, are generally long or can be designed to be so without construction difficulties or major expense. In level terrain, the surrounding terrain is considered to range from 0% to 8%.

Rolling Terrain

Rolling terrain is where the natural slopes consistently rise above and fall below the road grade and where occasional steep slopes offer some restrictions to horizontal and vertical roadway alignment. In rolling terrain, the surrounding terrain is generally considered to range from 8.1% to 15%.

Mountainous Terrain

Mountainous terrain is where longitudinal and transverse changes in the elevation of the terrain with respect to the road are abrupt and where benching and side hill excavations are frequently required to obtain acceptable horizontal and vertical alignment. In mountainous terrain, the surrounding terrain is considered to range over 15%.

Safety

TxDOT continues to develop additional strategies to incorporate safety into its system, contributing to the goal of eliminating traffic deaths statewide by 2050 (Road to Zero). The Department uses a number of initiatives related to developing and operating a safer highway system.

Highway Safety Improvement Program (HSIP)

HSIP is a federally funded program administered by TxDOT's Traffic Safety Division that allows highway safety improvements through strategic safety planning and performance measures. The HSIP requires states to develop and implement a Strategic Highway Safety Plan (SHSP). The SHSP identifies and analyzes highway safety problems and correction opportunities, including projects or strategies to evaluate the accuracy of data and prioritize the proposed improvements. The SHSP establishes target levels for five areas of fatal and serious injuries including the number and rate of fatalities, the number and rate of serious injuries, and trends for non-motorized fatalities and serious injuries. See *Highway Safety Improvement Project Manual* for more information and specific design criteria for HSIP projects.

Safety Analysis

Safety analysis uses crash data, traffic volume, and roadway geometrics. Various analytical tools and methods are available for analyzing potential safety impacts of potential improvements, including historical crash data analysis, AASHTO's Highway Safety Manual (HSM) Predictive Method, and a Crash Modification Factor (CMF) evaluation.

The historical crash data analysis involves 3 to 5 full calendar years of crash data with respect to characteristics such as severity, crash types, frequency, rates, patterns, clusters, and contributing factors. Crash diagrams such as heat maps, bar charts, and other maps graphically showing the crash emphasis locations are used to help interpret the data. A crash rate is the number of crashes that occur at a given location during a specified time period divided by measure of exposure. Crash rate is calculated per 100 million VMT using the following formula:

Crash Rate = 100,000,000 * $A/(365T * V * L)$
Where:
A = Number of reported crashes (in section or at location)
T= Time frame of the analysis, years
V = AADT, vehicles/day
L = Length of section, miles

The HSM Predictive Method provides procedures to analyze safety performance in terms of crash severity levels and collision types. Various spreadsheets and software have been developed to automate predictive analyses.

CMFs are used to estimate the anticipated impact of a countermeasure or mitigation on safety performance. A CMF is an index of the expected change in safety performance following a modification in traffic control strategy or design element. It can be used to estimate the safety effectiveness of a given strategy and compare the relative safety effectiveness of multiple strategies. The Crash Modification Factor Clearinghouse (www.cmfclearinghouse.org) offers a repository of CMFs.

The Design Division (DES) has a "Traffic Simulation and Safety Analysis" section whose purpose is to provide guidance and support for safety analysis. DES has developed multiple 'System Safety' tools which are used to estimate a safety score of a particular roadway segment by selecting various design elements. Use of the applicable tools should begin during project scoping to evaluate the safety impacts of design decisions. Refer to the Design Division webpage for additional information and guidance on all available tools.

Safety Analysis Data

TxDOT's Statewide Traffic Analysis and Reporting System (STARS) is the preferred resource for traffic data, while the Crash Records Information System (CRIS) is utilized for crash data.

Historical crash data is analyzed to identify potential safety problems that might be corrected. CRIS generates detailed crash data used to determine high crash locations, crash types, and contributing factors.

Statewide average crash rates are used in the crash rate analysis method and are useful to compare against the crash rates of a particular highway segment/intersection. TxDOT maintains ten years of crash data, available in summary reports.

TxDOT uses "crashes per year", level of crash severity, and crash type. Crash severity is classified as follows:

K – Fatal Injury
A – Suspected Serious Injury
B – Non-incapacitating injury
C – Possible injury
N – Not Injured
99 – Unknown

Where crash frequencies include wildlife-vehicle collision(s) as a contributing factor in the CRIS records, consult with the District Environmental Coordinator or with Environmental Affairs Division (ENV) to determine if a wildlife crossing structure could improve safety. The ENV Natural Resource Management Section and District Environmental Coordinators can provide information to conduct hot spot analysis and details on types of crossings, including schematics used within TxDOT and other states.

Additional Considerations

Turning Roadways and Intersection Corner Radii

Traffic volume and vehicle type influence the width and curvature of turning roadways and intersection corner radii. Minimum designs for turning roadways and various design vehicles are shown in Chapter 7, Section 7, Minimum Designs for Truck and Bus Turns.

Older Drivers and Older Pedestrians

Older drivers are a significant and growing segment of the road user population and should be accommodated in the design of road facilities to the extent practical. Research has shown that enhancements to the highway system to improve usability for older drivers and pedestrians can also improve the system for everyone.

The FHWA's *Handbook for Designing Roadways for the Aging Population* and AASHTO's *A Policy on Geometric Design of Highways and Streets* provide additional information for modifying geometric design elements and traffic control devices to better meet the needs and capabilities of older road users.

Section 4 — Sight Distance

Overview

This section provides descriptions and information on sight distance, one of several principal elements of design that are common to all types of highways and streets. Of utmost importance in highway design is the arrangement of geometric elements so that there is adequate sight distance for safe and efficient traffic operation assuming adequate light, clear atmospheric conditions, and drivers' visual acuity. For design, the following five types of sight distance are considered:

- Stopping Sight Distance;
- Decision Sight Distance;
- Passing Sight Distance;
- Intersection Sight Distance; and
- Sight Distance at Under-Crossings.

Stopping Sight Distance

Sight distance is the length of roadway ahead that is visible to the driver. The available sight distance on a roadway should be sufficiently long enough to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path.

Stopping sight distance is the sum of two distances:

- Brake reaction distance – the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied.
- Braking distance – the distance needed to stop the vehicle from the instant brake application begins on level terrain.

Approximately 90% of all drivers decelerate at rates greater than 11.2 ft/s^2. Such decelerations allow the driver to maintain steering control during the braking maneuver on wet surfaces. Therefore, 11.2 ft/s^2 is recommended as the deceleration threshold for determining stopping sight distance.

In computing and measuring stopping sight distance, the height of the driver’s eye is estimated to be 3.5-ft and the height of the object to be seen by the driver is 2.0-ft, equivalent to the taillight height of passenger car.

The calculated and design stopping sight distances are shown in Table 2-1.

The values given in Table 2-1 represent stopping sight distance on level terrain. As a general rule, the sight distance available on downgrades is larger than on upgrades, therefore, corrections for grade are usually unnecessary. An example where correction for grade might be applicable for stopping sight distance would be a divided roadway with independent design profiles in extreme rolling or mountainous terrain. AASHTO's *A Policy on Geometric Design for Highways and Streets*, provides additional information and suggested values for grade corrections in these rare circumstances.

Table 2-1: Stopping Sight Distance on Level Roadways

Design Speed (mph)	Brake Reaction Distance[1] (ft)	Braking Distance (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
15	55.1	21.6	76.7	80
20	73.5	38.4	111.9	115
25	91.9	60.0	151.9	155
30	110.3	86.4	196.7	200
35	128.6	117.6	246.2	250
40	147.0	153.6	300.6	305
45	165.4	194.4	359.8	360
50	183.8	240.0	423.8	425
55	202.1	290.3	492.4	495
60	220.5	345.5	566.0	570
65	238.9	405.5	644.4	645
70	257.3	470.3	727.6	730
75	275.6	539.9	815.5	820
80	294.0	614.3	908.3	910

Note:
1. Brake reaction distance predicated on a time of 2.5-s; deceleration rate 11.2-ft/s^2

Decision Sight Distance

Decision sight distance is the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source, recognize the source, select an appropriate speed and path, and initiate and complete the required maneuver safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to maneuver their vehicles at the same or reduced speed rather than to just stop, its values are substantially greater than stopping sight distance. Table 2-2 shows recommended decision sight distance values for various avoidance maneuvers.

Table 2-2: Recommended Decision Sight Distance Values

Avoidance Maneuver Decision Sight Distance (ft)					
Design Speed (mph)	**A[1]**	**B[2]**	**C[3]**	**D[4]**	**E[5]**
30	220	490	450	535	620
35	275	590	525	625	720
40	330	690	600	715	825
45	395	800	675	800	930
50	465	910	750	890	1030
55	535	1030	865	980	1135
60	610	1150	990	1125	1280
65	695	1275	1050	1220	1365
70	780	1410	1105	1275	1445
75	875	1545	1180	1365	1545
80	970	1685	1260	1455	1650

Notes:
1. Avoidance Maneuver A: Stop on rural road; t = 3.0s
2. Avoidance Maneuver B: Stop on urban road; t = 9.1s
3. Avoidance Maneuver C: Speed/path/direction change on rural road; t varies between 10.2 and 11.2s
4. Avoidance Maneuver D: Speed/path/direction change on suburban road; t varies between 12.1 and 12.9s
5. Avoidance Maneuver E: Speed/path/direction change on urban road; t varies between 14.0 and 14.5s
6. t = time in seconds

Examples of situations in which decision sight distance is preferred include the following:

- Interchange and intersection locations where unusual or unexpected maneuvers are required (such as exit ramp gore areas and left-side exits)
- Changes in cross-section such as lane drops
- Areas of concentrated demand where “visual noise” is present with competing sources of visual information, such as roadway elements, traffic elements, traffic control devices, and advertising signs

Locations along the roadway where a driver has stopping sight distance but not the extra response time provided by decision sight distance is identified as a “reduced decision zone”. Avoid placement of intersections within a reduced decision zone by relocating the intersection or by changing the grades to reduce the length of the reduced decision zone.

Passing Sight Distance

Passing sight distance is applicable only in the design of two-lane roadways (including two-way frontage roads) and therefore is presented in Chapter 3 Section 4, Two Lane Rural Highways, and Chapter 4 Section 6, Super 2 Highways.

Intersection Sight Distance

The operator of a vehicle approaching an intersection should have an unobstructed view of the entire intersection and an adequate view of the intersecting highway to permit control of the vehicle to avoid a collision. This visibility is referred to as intersection sight distance. When designing an intersection, the following factors should be taken into consideration:

- Adequate sight distance should be provided along both highway approaches and across corners;
- Gradients of intersecting highways should be as flat as practical on sections that are to be used for storage of stopped vehicles;
- Combination of vertical and horizontal curvature should allow adequate sight distance of the intersection;
- Traffic lanes and marked pedestrian crosswalks should be clearly visible at all times;
- Lane markings and signs should be clearly visible and understandable from a desired distance;
- Intersections should eliminate, relocate or modify conflict points to the extent allowable in order to improve safety; and
- Intersections should be evaluated for the effects of barriers, rails, and retaining walls on sight distance.

For selecting intersection sight distance, refer to AASHTO's *A Policy on Geometric Design for Highways and Streets*. Sight distance criteria are provided for the following types of intersection controls:

- Intersections with no control;
- Intersections with stop control on the minor road;
- Intersections with yield control on the minor road;
- Intersections with traffic signal control;
- Intersections with all-way stop control; and
- Left turns from the major road.

Sight Distance at Under-Crossings

Sight distance through a grade crossing should be at least the minimum stopping sight distance, or longer. Line of sight may be obstructed by an overpass structure and can limit the sight distance for the operator. Where practical, provide the minimum length of sag vertical curve at grade separated structures. Where economically feasible, for two-lane roadways, the passing sight distance should be maintained (see Chapter 3, Section 4 – Passing Sight Distances).

When the minimum sag vertical curve length based on headlight sight distance, as described in Section 6, is achieved for vertical clearances of 14' or higher, then Figure 2.1 and subsequent equations are not applicable. However, if any of the following conditions occur, then the under-crossing curve length as shown in Figure 2-1 and subsequent equations should be verified based on the applicable sight distance (SSD):

- The minimum sag vertical curve length based on SSD for headlights is not met
- The comfort control (lighting) criteria is used to establish the sag vertical curve length
- The vertical clearance is less than 14 feet

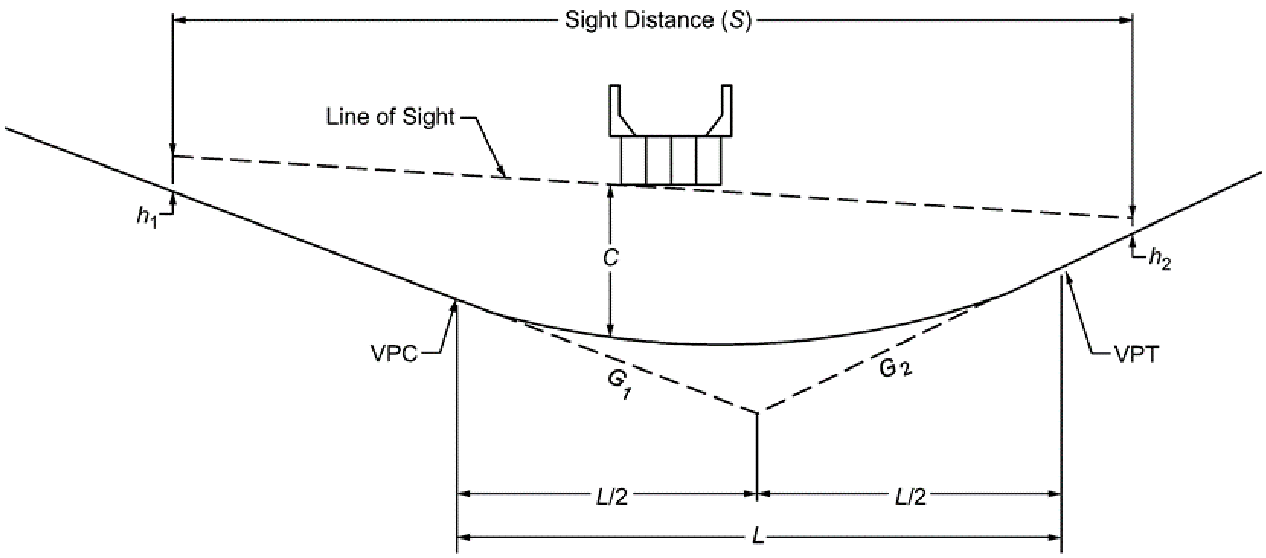


Figure 2-1. Sight Distance at Under-crossings.
Source: AASHTO's A Policy on Geometric Design of Highways and Streets

The general equations for sag vertical curve length at under-crossings are:

Case 1 – Sight distance greater than length of vertical Curve (*S*>*L*):

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

Where:

L = length of vertical curve, ft

S = sight distance, ft

C = vertical clearance, ft

h_1 = eye height, ft

h_2 = height of object, ft

A = algebraic difference in grades, percent

Case 2 – Sight distance less than length of vertical curve ($S<L$)

$$L = \frac{AS^2}{800\left[C - \frac{h_1 + h_2}{2}\right]}$$

Using an eye height of 8.0 ft for a truck driver and an object height of 2.0 ft for the taillights of a vehicle, the following equations can be derived:

Case 1 – Sight distance greater than length of vertical curve (S>L):

$$L = 2S - \frac{800(C-5)}{A}$$

Case 2 – Sight distance less than length of vertical curve (S<L):

$$L = \frac{AS^2}{800(C-5)}$$

Where:

L = length of vertical curve, ft

A = algebraic difference in grades, percent

S = sight distance, ft

C = vertical clearance, ft

h_1 = height of eye, ft

h_2 = height of object, ft

Section 5 — Horizontal Alignment

Overview

It is necessary to establish the proper relation between design speed and curvature when designing roadway alignment. The two basic elements of horizontal curves are:

- Curve Radius and
- Superelevation Rate.

General Considerations for Horizontal Alignment

There are a number of general considerations important for safe, smooth flowing, and aesthetically pleasing facilities. These practices as outlined below are particularly applicable to high-speed facilities.

- Flatter than minimum curvature for any particular design speed should be used where possible, while retaining the minimum guidelines for the most critical conditions;
- Alignment consistency should be sought. Sharp curves should not follow long tangents or a series of flat curves;
- Sharp curves should be avoided on long, high fills. It is difficult for drivers to perceive the extent of curvature and adjust their operation accordingly when the adjacent topography does not extend above the level of the roadway;
- Compound curves (two adjacent curves in the same direction with different radii) should be used with caution and should be avoided on mainlanes where conditions permit the use of simple curves. Where compound curves are used, the ratio of the flatter radius to the sharper radius should not exceed 3:2 (i.e., R_1 should not exceed $1.5R_2$). For intersections or other turning roadways (such as loops, connections, and ramps), this ratio may be increased to 2:1 (i.e., R_1 may be increased to $2R_2$);
- Reverse curves (two adjacent curves in opposite directions) on high-speed facilities should include a tangent section of sufficient length to provide adequate superelevation transition between the curves;
- Broken-back curves (two curves in the same direction with a short tangent between the curves) should be avoided if feasible. This configuration is unexpected by drivers, not pleasing in appearance, and more difficult for freight truck maneuverability; and
- Horizontal alignment and its associated design speed should be consistent with other design features and topography. Coordination with vertical alignment is discussed in Chapter 2, Section 6, Combination of Vertical and Horizontal Alignment.

Curve Radius

The design of roadway curves should be based on an appropriate relationship between design speed and curvature as well as their joint relationships with superelevation rate and side friction. The minimum radii of curves are important control values in designing for safe operation. Design guidance for low-speed urban facilities (45-mph or less) is shown in Table 2-3. Design guidance for curvature of high-speed (50-mph or greater) or non-urban facilities is shown in Table 2-4 and Table 2-5 for maximum superelevation (e_{max}) rates equal to 6 percent and 8 percent respectively.

For high-speed design conditions, the maximum allowable deflection angle without a horizontal curve is 30 minutes. For low-speed design conditions, the maximum allowable deflection angle without a horizontal curve is 1 degree.

Superelevation Rate

As a vehicle traverses a horizontal curve, it undergoes a centripetal acceleration that acts toward the center of the curve. Vehicle weight, roadway superelevation, and side friction between the tires and pavement surface sustain this acceleration. The equation that governs vehicle operation on a horizontal curve is:

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

Where:

- e = superelevation rate, ft/ft
- f = side friction factor
- V = vehicle speed, mph
- R = curve radius, ft

There are practical upper limits to the rate of superelevation. The Department normally uses a maximum superelevation rate of 6 percent. However, a maximum rate of 8 percent may be used where higher superelevation rates or sharper curves are desired. The recommended maximum for facilities where there is a regular occurrence of very-slow moving vehicles, whose operation might be affected by high superelevation rates is 6 percent. Use of 8 percent should be coordinated with the District Design Engineer prior to implementation and documented in the project files.

Superelevation Rates on Low-Speed Urban Facilities

Although superelevation is advantageous for traffic operations, various factors often combine to make its use impractical in many urban areas. These factors include the following:

- Wide pavement areas;
- Surface drainage considerations;

- Frequency of cross streets and driveways; and
- Need to meet the grade of adjacent property.

For these reasons, horizontal curves on low-speed urban facilities are frequently designed with normal crown. The centripetal acceleration, in this case, is counteracted solely with side friction. The term “normal crown” (NC) represents an equal downward pavement cross-slope, typically 2 percent, on each side of the axis of rotation.

Low-speed urban facilities should be designed using normal crown, such that superelevation is not necessary where practical. This is accomplished by using the negative e-values from Table 2-3. However, when superelevation is needed, a maximum superelevation rate of 4 percent should be used. This is accomplished by using the positive e-values from Table 2-3 to determine the superelevation rate for specific curvature and design speed conditions.

Table 2-3 shows the relationship of radius, superelevation rate, and design speed for low-speed urban facility design and should be used to evaluate existing conditions or the need for superelevation for proposed conditions on low-speed urban facilities. This table may also be used for design of detour alignments in constrained conditions.

- Example:

  Given a design speed of 35 mph and a 400 ft radius curve, Table 2-3 indicates an approximate superelevation rate of 2.4 percent should be used.

For a normal crown section, the negative e-value (the slope on the outside of the curve) will always be the controlling value for a given design speed.

- Example:

  Given a design speed of 45 mph and 1,050-ft radius curve, Table 2-3 indicates that a normal crown of 2 percent cross slope in each direction should be used.

Table 2-3: Minimum Radii and Superelevation Rates[1] for Low-Speed Urban Facilities

	Design Speed						
e (%)	**15 mph R (ft)**	**20 mph R (ft)**	**25 mph R (ft)**	**30 mph R (ft)**	**35 mph R (ft)**	**40 mph R (ft)**	**45 mph R (ft)**
-4.0[2]	54	116	219	375	583	889	1,227
-3.0[2]	52	111	208	353	544	821	1,125
-2.8[2]	51	110	206	349	537	808	1,107
-2.6[2]	51	109	204	345	530	796	1,089
-2.5[2,3]	51	109	203	343	527	790	1,080
-2.4[2]	51	108	202	341	524	784	1,071

Table 2-3: Minimum Radii and Superelevation Rates[1] for Low-Speed Urban Facilities

Design Speed							
e (%)	**15 mph R (ft)**	**20 mph R (ft)**	**25 mph R (ft)**	**30 mph R (ft)**	**35 mph R (ft)**	**40 mph R (ft)**	**45 mph R (ft)**
-2.2[2]	50	108	200	337	517	773	1,055
-2.0	50	107	198	333	510	762	1,039
-1.5[4,5]	49	105	194	324	495	736	1,000
-1.0[4,5]	48	103	189	316	480	711	964
-0.5[4,5]	48	101	185	308	467	688	931
0[5,6]	47	99	181	300	454	667	900
0.5[5]	46	97	177	293	441	646	871
1.0[5]	45	95	174	286	430	627	844
1.5[5]	45	94	170	279	419	610	818
2.0	44	92	167	273	408	593	794
2.2	44	91	165	270	404	586	785
2.4	44	91	164	268	400	580	776
2.6	43	90	163	265	396	573	767
2.8	43	89	161	263	393	567	758
3.0	43	89	160	261	389	561	750
3.2	43	88	159	259	385	556	742
3.4	42	88	158	256	382	550	734
3.6	42	87	157	254	378	544	726
3.8	42	87	155	252	375	539	718
4.0	42	86	154	250	371	533	711

Notes:

1. Computed using Superelevation Distribution Method 2. See AASHTO's A Policy on Geometric Design of Highways and Streets for the different types of Superelevation Distribution Methods.
2. Normal crown values beyond -2.0% should be used for surfaces such as gravel, crushed stone, and earth.
3. Areas with paved surfaces that receive more frequent rainfall events with high intensities and greater depths than other areas may use 2.5% normal crown.
4. For the purpose of evaluating existing conditions, normal crown values up to -1.5% may be used.
5. Values ranging from -1.5% to +1.5% should only be used in special circumstances such as intersections.
6. 0% is provided for information purposes only and should not be used for design.

Superelevation Rate on High-Speed or Non-Urban Facilities

Table 2-4 and 2-5 show superelevation rates (maximum 6 percent and 8 percent, respectively) for various design speeds and radii. These tables should be used for high-speed or non-urban facilities. For multi-lane facilities, particularly where wide medians are used, the radius applies to the inside edge of the innermost travel lane.

Table 2-4: Minimum Radii and Superelevation Rates[1]for High-Speed or Non-Urban Facilities, $e_{max} = 6\%$

	Design Speed													
e (%)	**15 mph R (ft)**	**20 mph R (ft)**	**25 mph R (ft)**	**30 mph R (ft)**	**35 mph R (ft)**	**40 mph R (ft)**	**45 mph R (ft)**	**50 mph R (ft)**	**55 mph R (ft)**	**60 mph R (ft)**	**65 mph R (ft)**	**70 mph R (ft)**	**75 mph R (ft)**	**80 mph R (ft)**
NC[3,4]	868	1,580	2,290	3,130	4,100	5,230	6,480	7,870	9,410	11,100	12,600	14,100	15,700	17,400
RC[3,4]	614	1,120	1,630	2,240	2,950	3,770	4,680	5,700	6,820	8,060	9,130	10,300	11,500	12,900
2.2	543	991	1,450	2,000	2,630	3,370	4,190	5,100	6,110	7,230	8,200	9,240	10,400	11,600
2.4	482	884	1,300	1,790	2,360	3,030	3,770	4,600	5,520	6,540	7,430	8,380	9,420	10,600
2.6	430	791	1,170	1,610	2,130	2,740	3,420	4,170	5,020	5,950	6,770	7,660	8,620	9,670
2.8	384	709	1,050	1,460	1,930	2,490	3,110	3,800	4,580	5,440	6,200	7,030	7,930	8,910
3.0	341	635	944	1,320	1,760	2,270	2,840	3,480	4,200	4,990	5,710	6,490	7,330	8,260
3.2	300	566	850	1,200	1,600	2,080	2,600	3,200	3,860	4,600	5,280	6,010	6,810	7,680
3.4	256	498	761	1,080	1,460	1,900	2,390	2,940	3,560	4,250	4,890	5,580	6,340	7,180
3.6	209	422	673	972	1,320	1,740	2,190	2,710	3,290	3,940	4,540	5,210	5,930	6,720
3.8	176	358	583	864	1,190	1,590	2,010	2,490	3,040	3,650	4,230	4,860	5,560	6,320
4.0	151	309	511	766	1,070	1,440	1,840	2,300	2,810	3,390	3,950	4,550	5,220	5,950
4.2	131	270	452	684	960	1,310	1,680	2,110	2,590	3,140	3,680	4,270	4,910	5,620
4.4	116	238	402	615	868	1,190	1,540	1,940	2,400	2,920	3,440	4,010	4,630	5,320
4.6	102	212	360	555	788	1,090	1,410	1,780	2,210	2,710	3,220	3,770	4,380	5,040
4.8	91	189	324	502	718	995	1,300	1,640	2,050	2,510	3,000	3,550	4,140	4,790
5.0	82	169	292	456	654	911	1,190	1,510	1,890	2,330	2,800	3,330	3,910	4,550
5.2	73	152	264	413	595	833	1,090	1,390	1,750	2,160	2,610	3,120	3,690	4,320
5.4	65	136	237	373	540	759	995	1,280	1,610	1,990	2,420	2,910	3,460	4,090
5.6	58	121	212	335	487	687	903	1,160	1,470	1,830	2,230	2,700	3,230	3,840
5.8	51	106	186	296	431	611	806	1,040	1,320	1,650	2,020	2,460	2,970	3,560

Table 2-4: Minimum Radii and Superelevation Rates[1]for High-Speed or Non-Urban Facilities, e_{max} = 6%

	Design Speed													
e (%)	**15 mph R (ft)**	**20 mph R (ft)**	**25 mph R (ft)**	**30 mph R (ft)**	**35 mph R (ft)**	**40 mph R (ft)**	**45 mph R (ft)**	**50 mph R (ft)**	**55 mph R (ft)**	**60 mph R (ft)**	**65 mph R (ft)**	**70 mph R (ft)**	**75 mph R (ft)**	**80 mph R (ft)**
6.0	39	81	144	231	340	485	643	833	1,060	1,330	1,660	2,040	2,500	3,050

Notes:

1. Computed using Superelevation Distribution Method 5. See AASHTO's A Policy on Geometric Design of Highways and Streets for the different types of Superelevation Distribution Methods.
2.
   - The term "NC" (normal crown) represents an equal downward cross-slope, typically 2%, on each side of the axis of rotation.
   - The minimum curve radii for normal crown are suitable up to 3.0%.
   - 3.0% normal crown should only be used when 3 or more lanes are sloped in the same direction.
   - 1.5% or flatter normal crown should only be used for the design of special circumstance, such as table-topping intersections, or the evaluation of existing conditions.
3. The term "RC" (reverse crown) represents a curve where the downward, or adverse, cross-slope should be removed by superelevating the entire roadway at the normal cross-slope rate.
4. For curve radii falling between normal crown and reverse crown, rather than interpolation a superelevation rate equal to the normal crown should typically be used.

Table 2-5: Minimum Radii and Superelevation Rates[1]for High-Speed or Non-Urban Facilities, e_{max} = 8%

	Design Speed													
e (%)	**15 mph R(ft)**	**20 mph R(ft)**	**25 mph R(ft)**	**30 mph R(ft)**	**35 mph R(ft)**	**40 mph R(ft)**	**45 mph R(ft)**	**50 mph R(ft)**	**55 mph R(ft)**	**60 mph R(ft)**	**65 mph R(ft)**	**70 mph R(ft)**	**75 mph R(ft)**	**80 mph R(ft)**
NC[2,4]	932	1,640	2,370	3,240	4,260	5,410	6,710	8,150	9,720	11,500	12,900	14,500	16,100	17,800
RC[3,4]	676	1,190	1,720	2,370	3,120	3,970	4,930	5,990	7,150	8,440	9,510	10,700	12,000	13,300
2.2	605	1,070	1,550	2,130	2,800	3,570	4,440	5,400	6,450	7,620	8,600	9,660	10,800	12,000
2.4	546	9,59	1,400	1,930	2,540	3,240	4,030	4,910	5,870	6,930	7,830	8,810	9,850	11,000
2.6	496	872	1,280	1,760	2,320	2,960	3,690	4,490	5,370	6,350	7,180	8,090	9,050	10,100
2.8	453	796	1,170	1,610	2,130	2,720	3,390	4,130	4,950	5,850	6,630	7,470	8,370	9,340
3.0	415	730	1,070	1,480	1,960	2,510	3,130	3,820	4,580	5,420	6,140	6,930	7,780	8,700

Table 2-5: Minimum Radii and Superelevation Rates[1] for High-Speed or Non-Urban Facilities, e_{max} = 8%

	Design Speed													
e (%)	15 mph R(ft)	20 mph R(ft)	25 mph R(ft)	30 mph R(ft)	35 mph R(ft)	40 mph R(ft)	45 mph R(ft)	50 mph R(ft)	55 mph R(ft)	60 mph R(ft)	65 mph R(ft)	70 mph R(ft)	75 mph R(ft)	80 mph R(ft)
3.2	382	672	985	1,370	1,820	2,330	2,900	3,550	4,250	5,040	5,720	6,460	7,260	8,130
3.4	352	620	911	1,270	1,690	2,170	2,700	3,300	3,970	4,700	5,350	6,050	6,800	7,620
3.6	324	572	845	1,180	1,570	2,020	2,520	3,090	3,710	4,400	5,010	5,680	6,400	7,180
3.8	300	530	784	1,100	1,470	1,890	2,360	2,890	3,480	4,140	4,710	5,350	6,030	6,780
4.0	277	490	729	1,030	1,370	1,770	2,220	2,720	3,270	3,890	4,450	5,050	5,710	6,420
4.2	255	453	678	955	1,280	1,660	2,080	2,560	3,080	3,670	4,200	4,780	5,410	6,090
4.4	235	418	630	893	1,200	1,560	1,960	2,410	2,910	3,470	3,980	4,540	5,140	5,800
4.6	215	384	585	834	1,130	1,470	1,850	2,280	2,750	3,290	3,770	4,310	4,890	5,530
4.8	193	349	542	779	1,060	1,390	1,750	2,160	2,610	3,120	3,590	4,100	4,670	5,280
5.0	172	314	499	727	991	1,310	1,650	2,040	2,470	2,960	3,410	3,910	4,460	5,050
5.2	154	284	457	676	929	1,230	1,560	1,930	2,350	2,820	3,250	3,740	4,260	4,840
5.4	139	258	420	627	870	1,160	1,480	1,830	2,230	2,680	3,110	3,570	4,090	4,640
5.6	126	236	387	582	813	1,090	1,390	1,740	2,120	2,550	2,970	3,420	3,920	4,460
5.8	115	216	358	542	761	1,030	1,320	1,650	2,010	2,430	2,840	3,280	3,760	4,290
6.0	105	199	332	506	713	965	1,250	1,560	1,920	2,320	2,710	3,150	3,620	4,140
6.2	97	184	308	472	669	909	1,180	1,480	1,820	2,210	2,600	3,020	3,480	3,990
6.4	89	170	287	442	628	857	1,110	1,400	1,730	2,110	2,490	2,910	3,360	3,850
6.6	82	157	267	413	590	808	1,050	1,330	1,650	2,010	2,380	2,790	3,240	3,720
6.8	76	146	248	386	553	761	990	1,260	1,560	1,910	2,280	2,690	3,120	3,600
7.0	70	135	231	360	518	716	933	1,190	1,480	1,820	2,180	2,580	3,010	3,480
7.2	64	125	214	336	485	672	878	1,120	1,400	1,720	2,070	2,470	2,900	3,370
7.4	59	115	198	312	451	628	822	1,060	1,320	1,630	1,970	2,350	2,780	3,250
7.6	54	105	182	287	417	583	765	980	1,230	1,530	1,850	2,230	2,650	3,120
7.8	48	94	164	261	380	533	701	901	1,140	1,410	1,720	2,090	2,500	2,970
8.0	38	76	134	214	314	444	587	758	960	1,200	1,480	1,810	2,210	2,670

Table 2-5: Minimum Radii and Superelevation Rates[1] for High-Speed or Non-Urban Facilities, e_{max} = 8%

	Design Speed													
e (%)	**15 mph R(ft)**	**20 mph R(ft)**	**25 mph R(ft)**	**30 mph R(ft)**	**35 mph R(ft)**	**40 mph R(ft)**	**45 mph R(ft)**	**50 mph R(ft)**	**55 mph R(ft)**	**60 mph R(ft)**	**65 mph R(ft)**	**70 mph R(ft)**	**75 mph R(ft)**	**80 mph R(ft)**

Notes:

1. Computed using Superelevation Distribution Method 5. See AASHTO's A Policy on Geometric Design of Highways and Streets for the different types of Superelevation Distribution Methods.
2.
   - The term "NC" (normal crown) represents an equal downward cross-slope, typically 2%, on each side of the axis of rotation.
   - The minimum curve radii for normal crown are suitable up to 3.0%.
   - 3.0% normal crown should only be used when 3 or more lanes are sloped in the same direction.
   - 1.5% or flatter normal crown should only be used for the design of special circumstance, such as table-topping intersections, or the evaluation of existing conditions.
3. The term "RC" (reverse crown) represents a curve where the adverse, or negative, cross-slope should be removed by superelevating the entire roadway at the normal cross-slope rate.
4. For curve radii falling between normal crown and reverse crown, a superelevation rate equal to the normal crown should typically be used.

Superelevation Transition Length

Superelevation transition is the general term denoting the change in cross slope from a normal crown section to the full superelevated section or vice versa. To meet the requirements of comfort and safety, the superelevation transition should occur over a length adequate for the usual travel speeds. Transition lengths should also account for potential future traveled way widening, including widening associated with the ultimate typical section in a schematic.

Desirable design values for length of superelevation transition are based on a given maximum relative gradient between profiles of the edge of traveled way and the axis of rotation. Table 2-6 shows recommended maximum relative gradient values. Transition length on this basis is directly proportional to the total superelevation, which is the product of the lane width and the change in cross slope.

Table 2-6: Maximum Relative Gradient (G) for Superelevation Transition

Design Speed (mph)	Maximum Relative Gradient[1] (%)	Equivalent Maximum Relative Slope (run:rise)
15	0.89	1:112
20	0.80	1:125
25	0.73	1:137
30	0.67	1:150
35	0.62	1:162
40	0.57	1:175
45	0.53	1:187
≥50	0.50	1:200

Note:

1. Maximum relative gradient for profile between edge of traveled way and axis of rotation.

Desirable transition length, L_{CT} can be calculated using the following equation:

$$L_{CT(des)} = \frac{(CS)(W)}{G}$$

Where:

L_{CT} = Calculated desirable transition length, ft
CS = Change in cross slope of superelevated pavement, percent
W = distance between the axis of rotation and the edge of traveled way, ft
G = maximum relative gradient (Table 2-6).

Example determinations of superelevation transition are shown in Figure 2-2.

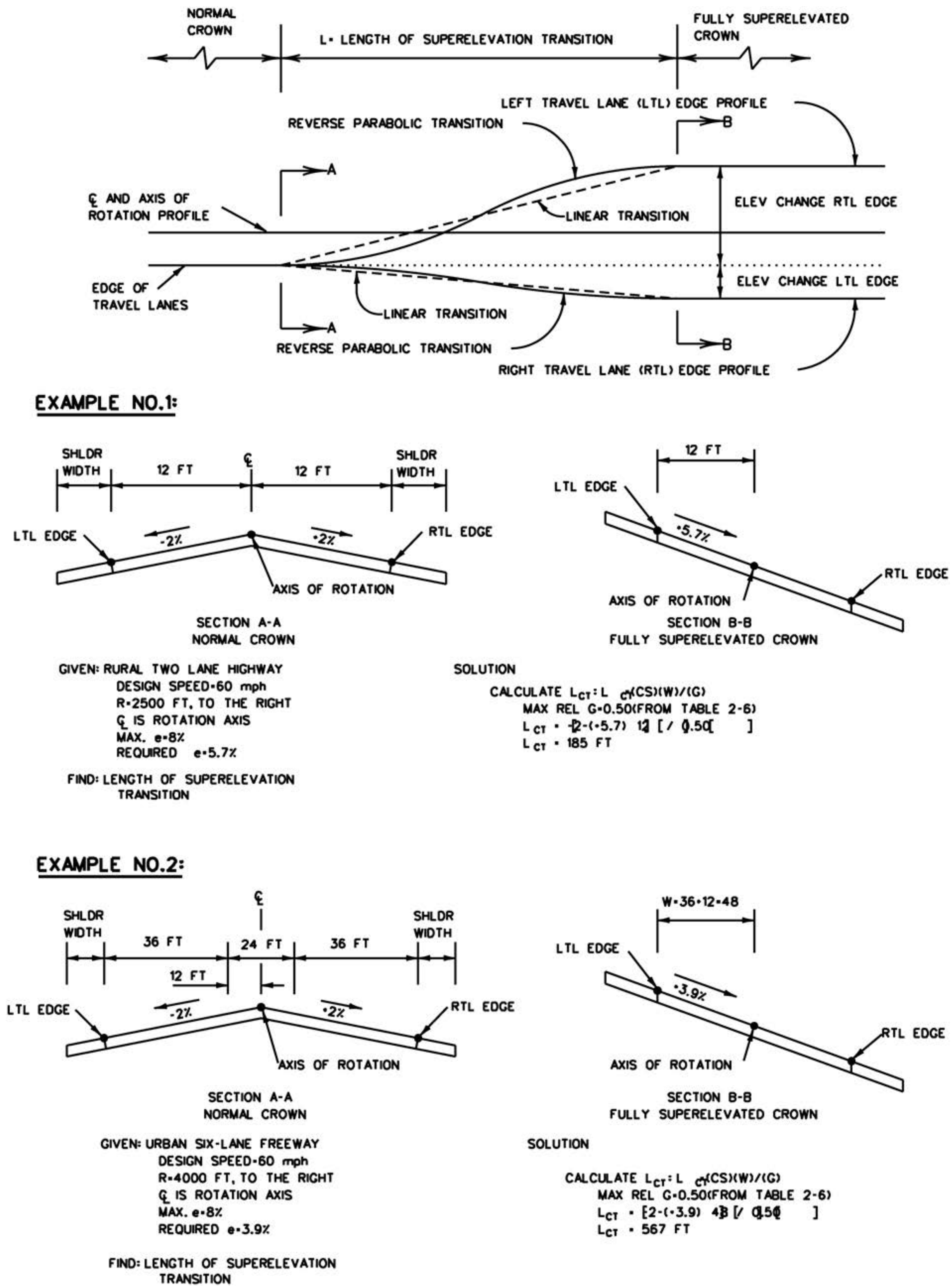


Figure 2-2. Determination of Length of Superelevation Transition.

As the number of lanes to be transitioned increases, the length of superelevation transition increases proportionately with the increased width. While strict adherence to the length (L_{CT}) calculation is desirable, the length for multi-lane facilities may become impractical for design purposes (e.g., drainage problems, avoiding bridges, accommodating merge/diverge condition). A minimum length (L_{CT}), can be calculated using adjustment factors as shown in Table 2-7, such that the transition length formula becomes:

$$L_{CT(min)} = \frac{(CS)(W)}{G}b$$

where “b” is defined in Table 2-7. In the case of one lane being rotated, “b” is 1.0, such that $L_{CT\ (min)} = L_{CT\ (des)}$

Table 2-7: Multilane Adjustment Factor[1]

Number of Lanes Rotated (n)	Adjustment Factor (b)
1.5	0.83
2	0.75
2.5	0.70
3	0.67
3.5	0.64

Note:
1. These adjustment factors are directly applicable to undivided facilities. For divided facilities where the axis of rotation is not the edge of traveled way, see AASHTO’s *A Policy on Geometric Design of Highways and Streets*.

Superelevation Transition Placement

The transition with respect to the termini of a simple (circular) curve should be placed to minimize lateral acceleration and the vehicle's lateral motion. The recommended allocation of superelevation transition on the tangent, preceding or following a curve, is provided on Table 2-8. For superelevation on bridge structures, it is preferred to begin/end superelevation at the bridge bent line. When spiral curves are present on an existing facility and alignment modifications aren't practical, refer to AASHTO's A Policy on Geometric Design of Highways and Streets for transition distribution.

Table 2-8: Portion of Superelevation Transition Located on the Tangent[1]

Design Speed (mph)	No. of Lanes Rotated			
	1.0	**1.5**	**2.0 - 2.5**	**3.0 - 3.5**
15 - 45	0.80	0.85	0.90	0.90
50 - 80	0.70	0.75	0.80	0.85

Note:

1. These values are recommendations based on prevailing research. A value between 0.7 and 0.9 for all speeds and rotated widths is considered acceptable. Refer to AASHTO's *A Policy on Geometric Design of Highways and Streets* for additional information.

Care must be exercised in designing the length and location of the superelevation transition. Pavement surfaces should be modeled to ensure proper drainage, especially near the high or low portions of Type I or III vertical curves (see Figure 2-5 for curve types). A plot of roadway contours may assist with the verification of grades and identification of drainage problems in areas of superelevation transition. Desirably, a minimum profile grade line (PGL) of 0.5 percent and minimum edge-of-pavement (EOP) profile grade of 0.2 percent (0.5 percent for curbed roadways) should be maintained throughout the transition section. At a minimum, either criterion should be met.

Whenever reverse curves are closely spaced and superelevation transition lengths overlap, transition lengths (L_{CT}) should be adjusted to ensure that roadway cross slopes are in the proper direction for each horizontal curve. For proposed construction of new facilities, the tangent section between reverse curves should be of sufficient length such that minimum transition lengths for each transition do not overlap.

Superelevation Transition Type

Linear or reverse parabolic transitions may be used for attaining superelevation. Where appearance is a factor (e.g., curbed sections and retaining walls) use of reverse parabolic is recommended. This produces an outer edge profile that is smooth, undistorted, and pleasing in appearance. However, for bridges, linear transitions are generally preferred for constructability, ride quality, and lower cost. Notate the transition type in the plans to ensure the transition is properly constructed.

Figure 2-2 shows reverse parabolic and linear transitions over the full length of the transition. Refer to AASHTO's *A Policy on Geometric Design of Highways and Streets* for alternative methods for developing smooth-edge profiles over the length of the transition.

Sight Distance on Horizontal Curves

Where an object off the pavement restricts sight distance, such as a bridge pier, bridge railing, median barrier, retaining wall, building, cut slope or natural growth, the minimum radius of curvature is determined by the stopping sight distance.

The following equation applies only to circular curves longer than the stopping sight distance (S<L) for the pertinent design speed. For example, with a 50-mph design speed and a curve with a 1,150-ft radius, a clear sight area with a horizontal sight line offset (HSO) of approximately 20-ft is needed for stopping sight distance.

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

Where:

HSO = horizontal sight line offset, ft
S = stopping sight distance (Table 2-1), ft
R = radius at centerline of inner most travel lane, ft

This method for calculating HSO is only exact when both the vehicle and sight obstruction are located within the horizontal curve. When the vehicle or sight obstruction are located outside of the horizontal curve (i.e. S>L) this method will result in an HSO slightly larger than required. In many instances the resulting additional clearance will not be significant. In some cases, the design should be checked either by using graphical procedures or computational methods to verify HSO. NHCRP 910 provides computational methods for verifying HSO.

In cases where complex geometries or discontinuous objects cause sight obstructions, graphical methods may be useful in determining available sight distance and associated offset requirements. Graphical methods may also be used when the circular curve is shorter than the stopping sight distance.

To check horizontal sight distance on the inside of a curve graphically, sight lines equal to the required sight distance on horizontal curves should be reviewed to ensure that obstructions such as buildings, hedges, barrier railing, and high ground do not restrict the sight distance required in either direction. Figure 2-3 illustrates a graphical approach to determining horizontal sight distance in a curve.

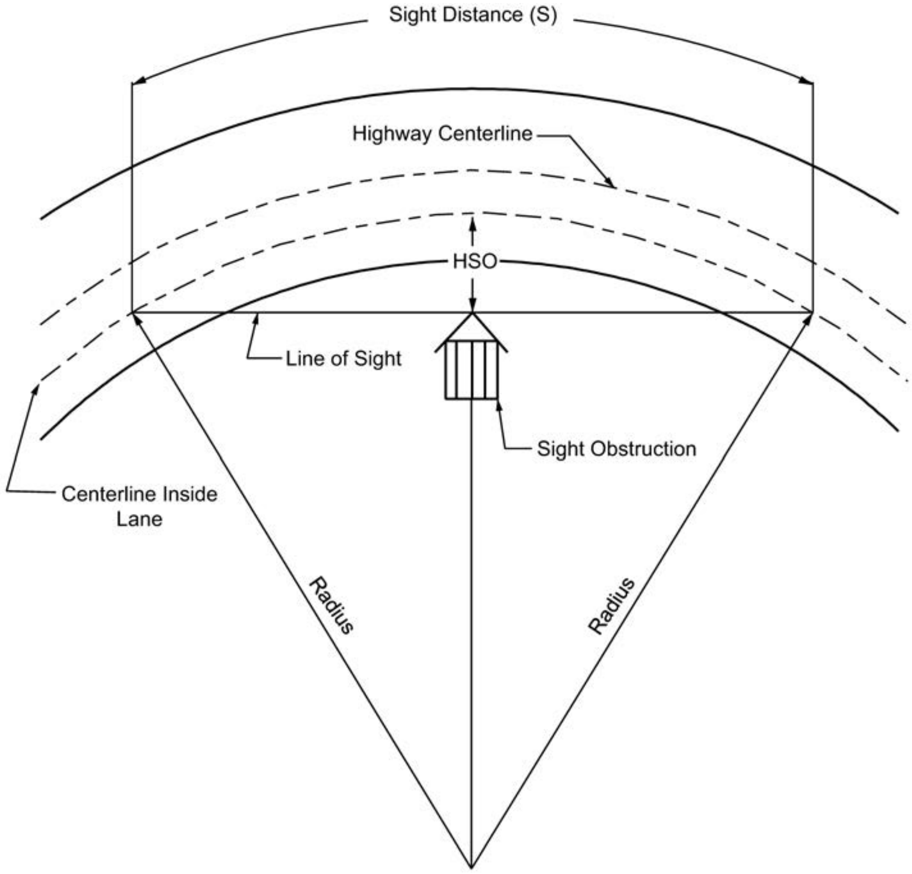


Figure 2-3. Diagram Illustrating Components for Determining Horizontal Sight Distance. Source: AASHTO's A Policy on Geometric Design of Highways and Streets

Where sufficient stopping sight distance is not available because a railing, longitudinal barrier or other features constitutes a sight obstruction, alternative designs should be considered. Potential alternatives include: (1) increasing the offset to the obstruction or (2) increasing the radius. However, the alternative should not incorporate a shoulder width on the inside of the curve in excess of 12-ft because of the concern that drivers will use wider shoulders as a passing or travel lane.

Section 6 — Vertical Alignment

Overview

The two basic elements of vertical alignment are:

- Grades; and
- Vertical Curves.

Grades

The effects of rate and length of grade are more pronounced on the operating characteristics of trucks than on passenger cars and thus may introduce undesirable speed differentials between the vehicle types. The term "critical length of grade" is used to indicate the maximum length of a specified ascending gradient upon which a loaded truck can operate without an unreasonable reduction in speed (commonly 10 mph). Figure 2-4 shows the relationship of percent upgrade, length of grade, and speed reduction for a representative truck (200-lb/hp weight/power ratio) with an entering speed of 70 mph. Where critical length of grade is exceeded for two-lane highways, climbing lanes should be considered as discussed in the TRB's *Highway Capacity Manual*. When the critical length of grade values shown in Figure 2-4 are exceeded, consider the effect on the upstream traffic of adding climbing lanes (i.e. two-lane facilities on direct connectors should be considered when a one-lane facility would significantly reduce the LOS).

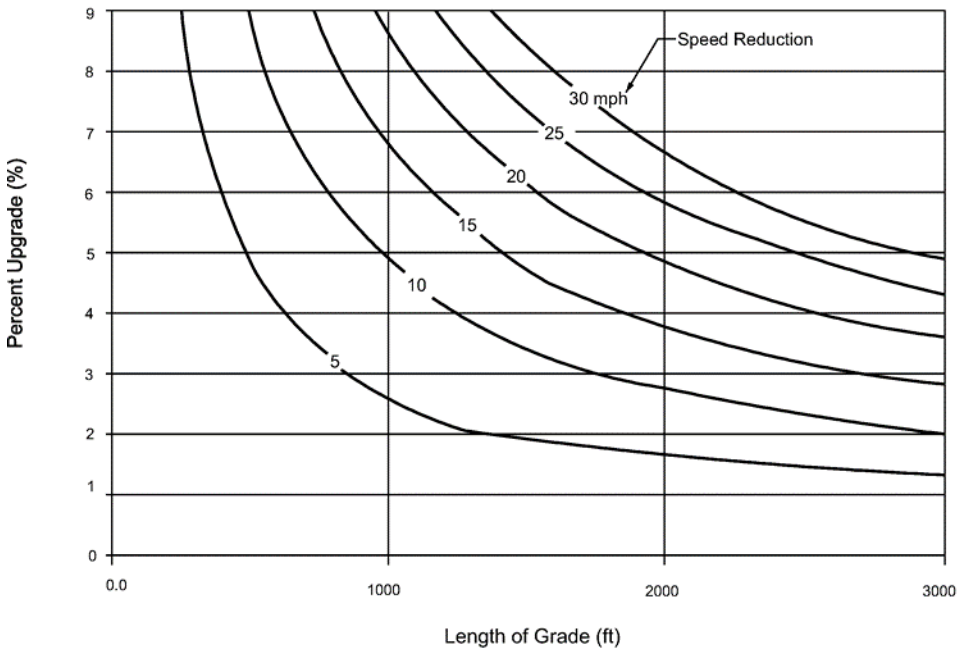


Figure 2-4. Critical Lengths of Grade for Design, Assumed Typical Heavy Truck of 200-lb/hp, Entering Speed = 70 mph.
Source: AASHTO's A Policy on Geometric Design of Highways and Streets

Table 2-9 summarizes the maximum grade controls in terms of design speed. See Chapter 2 Section 3, Terrain for defining level, rolling, and mountainous conditions. Generally, maximum design grade should be used infrequently rather than as a value to be used in most cases. However, for certain cases such as urban freeways, a maximum value may be applied in blanket fashion on interchange and grade separated approaches.

Table 2-9: Maximum Grades

Functional Classification	Type of Terrain	Design Speed (mph)													
		15	20	25	30	35	40	45	50	55	60	65	70	75	80
Urban and Suburban															
Local[1]	All	8	8	8	8	8	8	8							
Collector	Level	9	9	9	9	9	9	8	7	7	6				
	Rolling	12	12	12	11	10	10	9	8	8	7				
Arterial	Level				8	7	7	6	6	5	5				
	Rolling				9	8	8	7	7	6	6				
Freeway	Level								4	4	3	3	3	3	3
	Rolling								5	5	4	4	4	4	4
Rural:															
Local	Level	9	8	7	7	7	7	7	6	6	5				
	Rolling	12	11	11	10	10	10	9	8	7	6				

Table 2-9: Maximum Grades

Functional Classification	Type of Terrain	Design Speed (mph)													
		15	20	25	30	35	40	45	50	55	60	65	70	75	80
Collector	Level		7	7	7	7	7	7	6	6	5				
	Rolling		10	10	9	9	8	8	7	7	6				
Arterial	Level						5	5	4	4	3	3	3	3	3
	Rolling						6	6	5	5	4	4	4	4	4
Freeway	Level								4	4	3	3	3	3	3
	Rolling								5	5	4	4	4	4	4

Note:
1. Less than 15% maximum can be used in very constrained conditions. Flatter gradients should be used where practical and grades less than 5% are desirable.

Flat or level grades on uncurbed pavements are satisfactory when the pavement is adequately crowned to drain the surface water laterally and superelevation transitions are not co-located (see Combination of Horizontal and Vertical Alignment section). When roadside ditches are required and follow the grade of the roadway, the roadway profile grade should seldom be less than 0.5 percent for unpaved ditches and 0.25 percent for lined channels. When these roadway profile grade conditions are not met, special ditch profiles, independent of the roadway profile may be required to ensure positive ditch drainage. For curbed pavements, a minimum roadway profile grade of 0.3 percent should be provided to facilitate surface drainage. Joint analyses of rainfall frequency and duration, longitudinal grade, cross slope, curb inlet type, and spacing of inlets or discharge points are usually required so the width of water on the pavement surface during likely storms does not unduly interfere with traffic. Criteria for water ponding for various functionally classified roadways are contained in the *Hydraulic Design Manual*.

Vertical Curves

Vertical curves provide gradual changes between tangents of different grades. The simple parabola shown in Figure 2-5 is used in the highway profile design of vertical curves.

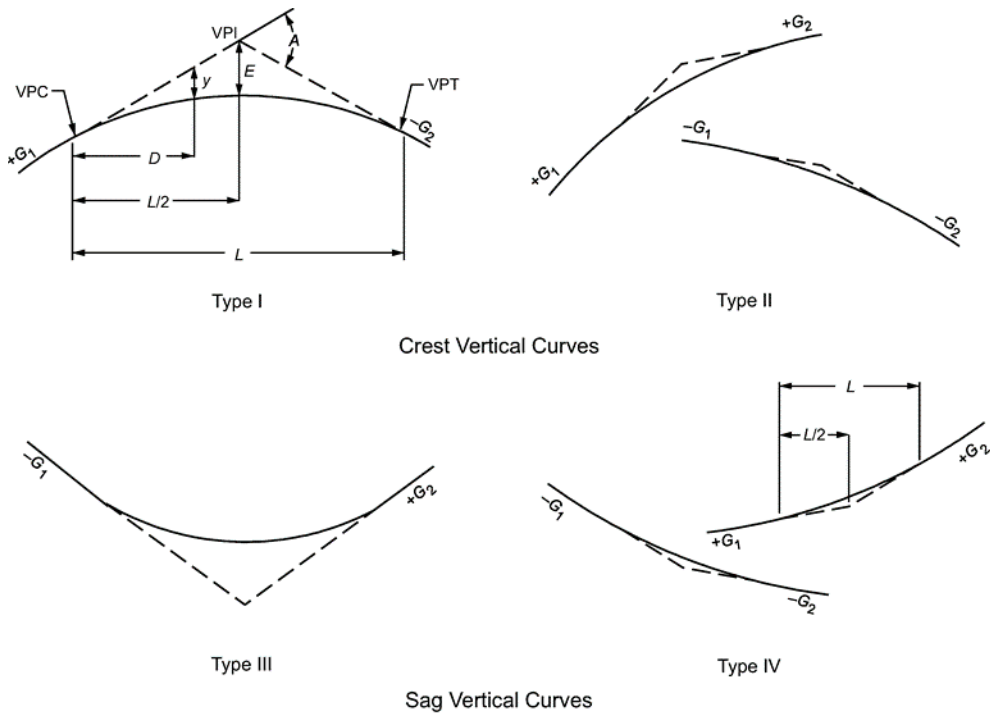


Figure 2-5. Types of Vertical Curves.
Source: AASHTO's A Policy on Geometric Design of Highways and Streets

For vertical curve discussion purposes, the following parameters are defined:

$$K = \frac{L}{A}$$

$$E = \frac{AL}{800}$$

$$y = \frac{4D^2E}{L^2} \quad \text{OR} \quad y = \frac{D^2A}{200L}$$

Where:

G_1 and G_2 = tangent grades, percent
K = length of vertical curve per percent change in A (also known as the design control)
A = algebraic difference in grades, percent;
L = length of vertical curve, ft
E = vertical offset at the VPI, ft
y = ordinate from tangent to curve, ft
D = distance from nearest VPC or VPT to any point on curve, ft

The minimum lengths of crest and sag vertical curves for different values of A to provide the stopping sight distances for each design speed are shown in Figure 2-6 and Figure 2-7, respectively.

The solid lines give the minimum vertical curve lengths on the basis of rounded values of K. These lengths represent minimum values based on design speed.

A dashed curve crossing the solid lines indicates where sight distance (S) equals length of vertical curve (L) (i.e. S = L). Note that to the right of the S = L line, the value of K is a simple and convenient expression of the design control. For each design speed, this single value is a positive number that is indicative of the rate of vertical curvature. The design control in terms of K covers all combinations of A and L for any one design speed; thus, A and L need not be indicated separately in a tabulation of the design values. The selection of design curves is facilitated because the length of curve is equal to K times the algebraic difference in grades in percent, L = KA. Conversely, the checking of curve design is simplified by comparing all curves with the design value for K.

Where S is greater than L, the values plot as a curve (as shown by the dashed curve extension for 45 mph). For small values of A, the vertical curve lengths are zero because the sight line passes over the apex. Since this relationship does not represent desirable design practice except in limited conditions (see discussion on Grade Change Without Vertical Curves), a minimum length of vertical curve is shown. Attention should be given where there are successive vertical curves.

Desirable minimum lengths of vertical curves (both crest and sag) are three times the design speed in miles per hour (L_{min} = 3 V). However, this minimum length is not considered a design control (i.e., a design waiver would not be required for these minimum length values if the minimum stopping sight distance for the relevant design speed is met).

For sag vertical curves, at least four different criteria for establishing the lengths of sag vertical curves are recognized to some extent. These are (1) headlight sight distance, (2) passenger comfort, (3) drainage control, and (4) general appearance.

Generally, a sag vertical curve should be long enough that the light beam distance is nearly the same as the stopping sight distance, especially at intersections located within the vicinity of the sag curve. Accordingly, it is appropriate to use stopping sight distances for different design speeds to establish sag vertical curve lengths. The resulting sag vertical curves for the recommended stopping sight distances for each design speed are shown in Figure 2-7 with the solid lines representing the rounded K values.

There is a level point on a vertical curve which can affect drainage; particularly on curbed facilities. Typically, there is no difficulty with drainage on highways if the curve is sharp enough so that a minimum grade of 0.30 percent is reached at a point about 50-ft from the crest or sag. This corresponds to a K value of 167 which is plotted in Figure 2-6 and Figure 2-7 as the drainage threshold. All combinations above or to the left of this line satisfy the drainage criterion. The combinations below and to the right of this line involve flatter vertical curves. Special attention is needed in these cases to ensure proper pavement drainage. It is not intended that these values be considered a design maximum, but merely a value beyond which drainage should be more carefully designed.

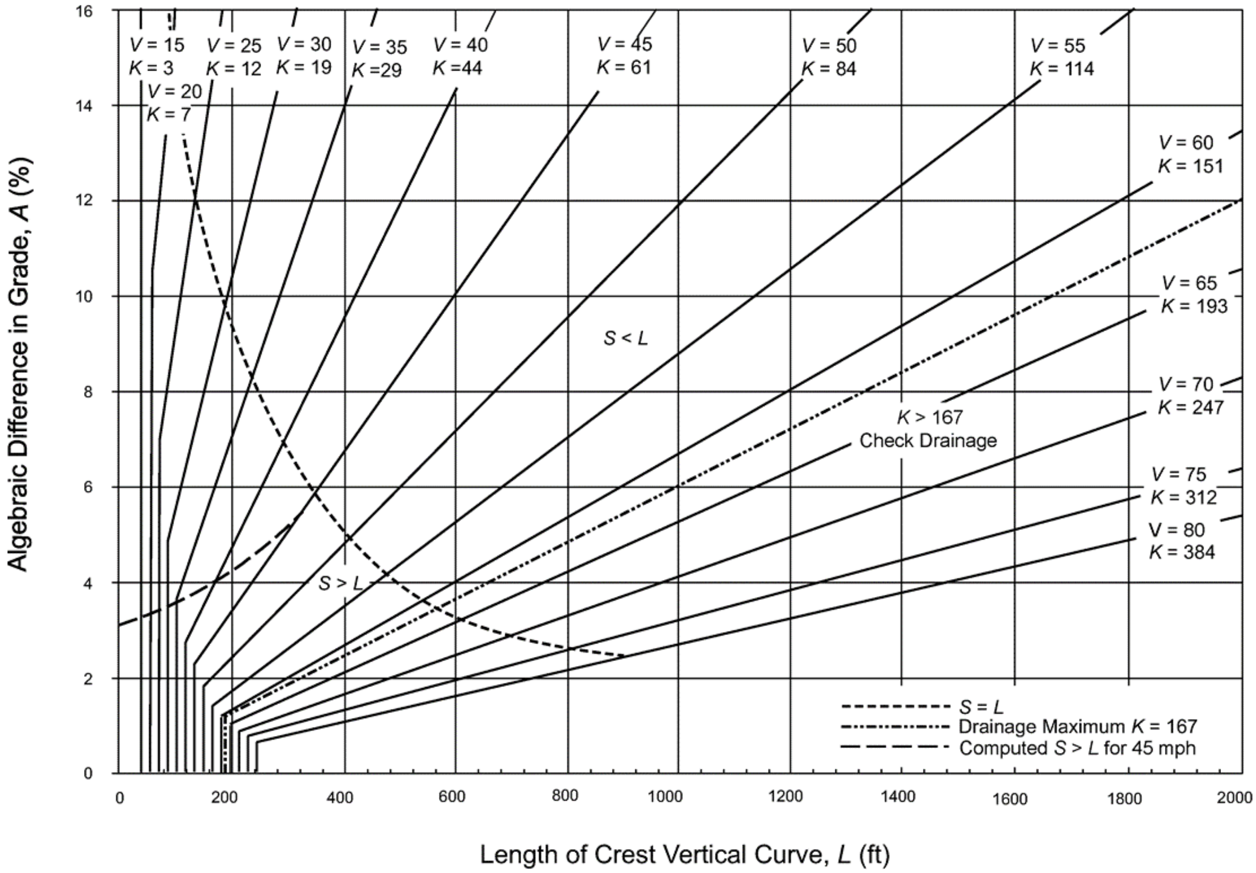


Figure 2-6. Design Controls for Crest Vertical Curves.
Source: AASHTO's A Policy on Geometric Design of Highways and Streets

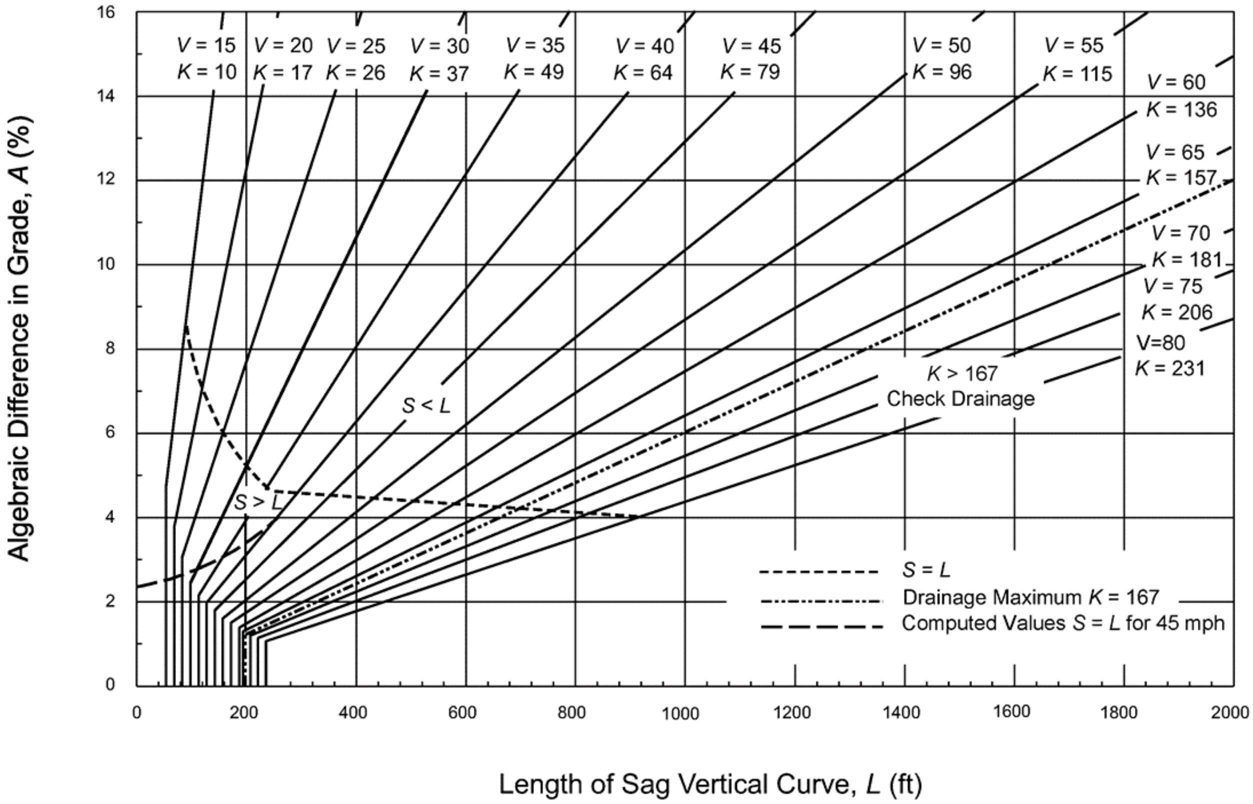


Figure 2-7. Design Controls for Sag Vertical Curves.
Source: AASHTO's A Policy on Geometric Design of Highways and Streets

Table 2-10 provides minimum vertical curve lengths and associated K as a function of A. Longer curves should be used wherever practical.

Table 2-10. Minimum K and L as a Function of A

Speed (mph)	Crest Vertical Curves			Sag Vertical Curves		
	A (%)	Min K	Min. Length of Curve (ft)	A (%)	Min K	Min. Length of Curve (ft)
15	≥ 15.000	3	-	> 4.500	10	-
	< 15.000	-	45	≤ 4.500	-	45
20	≥ 8.571	7	-	> 3.529	17	-
	< 8.571	-	60	≤ 3.529	-	60
25	≥ 6.250	12	-	> 2.884	26	-
	< 6.250	-	75	≤ 2.884	-	75
30	≥ 4.736	19	-	> 2.432	37	-
	< 4.736	-	90	≤ 2.432	-	90
35	≥ 3.620	29	-	> 2.142	49	-
	< 3.620	-	105	≤ 2.142	-	105

Table 2-10. Minimum K and L as a Function of A

Speed (mph)	Crest Vertical Curves			Sag Vertical Curves		
	A (%)	Min K	Min. Length of Curve (ft)	A (%)	Min K	Min. Length of Curve (ft)
40	≥ 2.727	44	-	> 1.875	64	-
	< 2.727	-	120	≤ 1.875	-	120
45	≥ 2.213	61	-	> 1.708	79	-
	< 2.213	-	135	≤ 1.708	-	135
50	≥ 1.785	84	-	> 1.562	96	-
	< 1.785	-	150	≤ 1.562	-	150
55	≥ 1.447	114	-	> 1.434	115	-
	< 1.447	-	165	≤ 1.434	-	165
60	≥ 1.192	151	-	> 1.323	136	-
	< 1.192	-	180	≤ 1.323	-	180
65	≥ 1.010	193[1]	-	> 1.242	157	-
	< 1.010	-	195	≤ 1.242	-	195
70	≥ 0.850	247[1]	-	> 1.160	181[1]	-
	< 0.850	-	210	≤ 1.160	-	210
75	≥ 0.721	312[1]	-	> 1.092	206[1]	-
	< 0.721	-	225	≤ 1.092	-	225
80	≥ 0.625	384[1]	-	> 1.038	231[1]	-
	< 0.625	-	240	≤ 1.038	-	240

Note:
1. Exceeds recommended drainage maximum of K > 167. Ensure proper pavement drainage.

Because cost and energy conservation considerations are factors in operating continuous lighting systems, headlight sight distance should be generally used in the design of sag vertical curves. “Comfort control criteria” is approximately 50 percent of the sag vertical curve lengths required by headlight distance and should be reserved for special use. Instances where the comfort control criteria may be appropriately used include ramp profiles where safety lighting is provided and for economic reasons in cases where an existing element, such as a structure not ready for replacement, controls the vertical profile. Comfort control criteria should be used sparingly on continuously lighted facilities since local, outside agencies often maintain and operate these systems and operations could be curtailed in the event of energy shortages. A design waiver is not required if the sag vertical curve length meets comfort control criteria and lighting is provided.

Refer to Figure 2-1 for more information about sag vertical curve design to ensure that overhead sight obstructions such as structures for overpassing roadways, overhead sign bridges, and tree crowns do not reduce stopping sight distance below the appropriate minimum value.

Grade Change without Vertical Curves

Roadway profiles should be designed using vertical curves at vertical points of intersection and intersecting streets where practical. Designing a sag or crest vertical point of intersection without a vertical curve is generally acceptable where the grade difference (A) is:

- 1 percent or less for design speeds less than or equal to 45 mph; and
- 0.5 percent or less for design speeds greater than 45 mph.

  When a grade change without vertical curve is specified, the construction process typically results in a short vertical curve being built (i.e., the actual point of intersection is "smoothed" in the field). Conditions where grade changes without vertical curves are not recommended include:

- Free flow intersections (without stop or yield control);
- Bridges (including bridge ends);
- Direct-traffic culverts; and
- Other locations requiring carefully detailed grades.

The minor approach of an intersection with yield or stop control may be constructed without vertical curves, provided that a grade change does not adversely affect local vehicle operations. A review of local traffic characteristics should be performed to identify a target design vehicle for each intersection for determining an appropriate grade change without a vertical curve. Typically, a grade change of 3 percent or less accommodates most vehicles. However, in highly constrained conditions, a grade change of up to 8 percent may operate effectively depending on local site conditions. A design waiver or design exception will not be required if stopping sight distance is met. Use of this criterion should be coordinated with the District Design Engineer prior to implementation and documented in the project files.

Vertical Alignment at Railroad Crossings

It is desirable that the intersection of highway and railroad be as level as possible from the stand point of sight distance, rideability, and braking and acceleration distances. If a railroad crossing is located at the peak of a vertical curve on the highway, the curve should be adequately flat to avoid hanging-up of vehicles and of sufficient length to ensure an adequate view of the crossing consistent with the highway design or operating speed. Refer to AASHTO's A Policy on Geometric Design for Highways and Streets for additional information.

Vertical Alignment at Intersections

Adequate intersection sight distance should be provided at intersecting roads. At a minimum, stopping sight distance (measured from the edge of travel way of the intersecting facility) must be provided. The grades of intersecting roads should be as level as practical. Approaching an intersection, drivers might misjudge the effect of steep grades on stopping or accelerating distances. Approach grades steeper than 3 percent should be avoided within the stopping sight distance of the intersection. Where conditions make such designs impractical, grades should not exceed 6 percent.

At intersections where marked or unmarked crosswalks are present, the roadway profile grade within the crosswalks should be limited so that the crosswalk is accessible and usable by all individuals, including those with disabilities. The roadway profile grade across a crosswalk must not exceed:

- 2 percent for intersections with yield or stop control; or
  - Intersections with yield or stop control are those in which a vehicle must always slow or stop before proceeding through the intersection; for example an intersection with a yield sign or stop sign.
- 5 percent for intersections without yield or stop control.
  - Intersections without yield or stop control are those in which a vehicle may proceed through the intersection without slowing or stopping; for example an intersection with a traffic signal that has a green cycle phase, or an intersection without a yield sign or stop sign.

Accessibility accommodations should be considered when determining appropriate profile grades for all future crosswalk locations. Refer to Chapter 7 Section 3, Pedestrian Facilities for additional guidance.

It is generally desirable to provide a smooth junction at intersecting streets. To accomplish this, the profile, cross slopes, and crowns of both roads are warped to create a common plane at the intersection. This technique is commonly referred to as “table-topping.” Design features including but not limited to proper drainage and pedestrian use should be carefully considered when table-topping an intersection. When table-topping of an intersection is not feasible and minor roadway traffic is expected to stop or travel slowly through the intersection, grades should generally favor the major roadway to provide a smooth profile. This is accomplished by warping the grades on the minor roadway to fit the cross-section of the major roadway. Similarly, for intersections contained within a horizontal curve with superelevation, an attempt should be made to match the superelevated section through the intersection.

Additional profiling of intersection features (e.g., curb, median islands, and curb ramps) may be required to ensure proper drainage and a smooth intersection surface. Contours at appropriate intervals (e.g., 1-ft major and 0.25-ft minor contours) and key curb elevations should be shown in the plans for proper construction of such intersections. Low points near intersections must be designed to prevent water ponding within pedestrian access routes. A minimum pavement edge profile grade of 0.30% should be maintained along corner radii of curbed intersections.

Vertical Clearance

Minimum vertical clearance for structures should be calculated from the point on the roadway with least clearance including the outside edge of one shoulder to the other and on both sides of the structure being passed under to verify all vehicles can clear the structure at any point along the structure including travel lanes and shoulders. Required vertical clearances also allow for potential future overlays. Table 2-11 shows the minimum vertical clearances required. Reference Chapter 1, Section 2 for additional information on vertical clearance design exceptions, and THFN design deviations.

Table 2-11. Vertical Clearance Requirements[1]

Functional Classification	Vehicle Overpasses[2] (ft)	Overhead Sign Structures and Bicycle / Pedestrian Overpasses (ft)
Local/Collector/Arterial	16.5[3]	17.5
Freeway[4]	16.5	17.5
Texas Highway Freight Network (THFN)[5]	18.5	19.5

Notes:
1. Vertical clearance requirements include an additional 0.5-ft. to accommodate future resurfacing as required by AASHTO's *A Policy on Geometric Design of Highways and Streets*.
2. Intersecting roadway goes over the functionally classified roadway.
3. Exceptional cases near as practical to 16.5 ft but never less than 14.5 ft.
4. See Chapter 3, Section 6 for additional vertical clearance guidance on freeways.
5. Additional vertical clearance requirements if project criteria specified in Ch. 3, Section 8 are met.

During construction, refer to the Bridge Design Manual for requirements on maximum time between erection of girders and placement of deck for spans over active lanes to avoid over-height impacts. At a minimum, during construction, bridge vertical clearances and all other obstructions above traffic, including temporary works, must be 14.5-ft. 15-ft is preferable to provide a sufficient buffer between allowed and actual heights to avoid over height impacts.

Combination of Vertical and Horizontal Alignment

Due to the near permanent nature of roadway alignment once constructed, it is important that the proper alignment be selected consistent with design speed, existing and future roadside development, subsurface conditions, topography, etc. The following factors are general considerations in obtaining a proper combination of horizontal and vertical alignment:

- The design speed of both vertical and horizontal alignment should be compatible with longer vertical curves and flatter horizontal curves than dictated by minimum values;
- Design speed should be compatible with topography with the roadway fitting the terrain where feasible;

- Alignment should be as flat as possible near intersections to maximize intersection sight distance;
- For rural divided facilities:
  - Independent mainlane profiles are often more aesthetic and economical;
  - Where used on non-controlled access facilities with narrow medians, median openings should be placed at locations that minimize crossover grades and provide adequate sight distance for vehicles stopped therein;
  - When designing independent vertical and horizontal profiles on divided facilities, considerations should be given to the impact these profiles may have on future widening into the median; and
- For two-lane rural highways and Super 2 highways the need for safe passing sections at frequent intervals should be carefully considered in developing horizontal and vertical alignments.

Refer to AASHTO's A Policy on Geometric Design of Highways and Streets for examples of good and poor practices in alignment and profile relationships.

Section 7 — Cross-Sectional Elements

Overview

This section includes information on the following cross-sectional design elements:

- Pavement Cross Slope;
- Median Design;
- Lane Widths;
- Shoulder Widths;
- Pavement Taper Lengths;
- Curb and Curb with Gutters;
- Roadside Design;
- Slopes and Ditches;
- Lateral Offset to Obstructions; and
- Clear Zone.

Pavement design is covered in TxDOT's *Pavement Manual*.

Pavement Cross Slope

The operating characteristics of vehicles on crowned pavements are such that for cross slopes up to 2 percent, the effect on steering is barely perceptible. A reasonably steep lateral slope is desirable to minimize water ponding on flat sections of uncurbed pavements due to imperfections or unequal settlement. With curbed pavements, a sufficiently steep cross slope is desirable to contain the flow of water adjacent to the curb. The recommended pavement cross slope for usual conditions is 2 percent. In areas of frequent rainfall events with high intensities, steeper cross slopes may be used as discussed in AASHTO's *A Policy on Geometric Design of Highways and Streets*.

Highways with three or more lanes inclined in the same direction should desirably have an increasing cross slope as the distance from the crown line increases to facilitate pavement drainage. In these cases, the first two lanes adjacent to the crown line may be sloped flatter than normal-typically at 1.5 percent but not less than 1 percent. The cross slope of each successive pair of lanes (or single lane if that is the outside lane) outward from the crown should be increased by 0.5 to 1.0 percent from the cross slope of the adjacent lane. A cross slope should not normally exceed 3 percent on a tangent alignment unless there are three or more lanes in one direction of travel. In areas of intense rainfall and where three or more lanes are provided in one direction of travel, the maximum cross slope should be limited to 4 percent.

On bridge structures with three or more lanes in one direction, maintain a constant slope of 2.5 percent, transitioning before and after the bridge accordingly.

For tangent sections on divided highways, each pavement should have a uniform cross slope with the high point at the edge nearest the median. Although a uniform cross slope is preferable, on rural sections with a wide median, the high point of the crown is sometimes placed at the centerline of the pavement with cross slopes from 1.5 to 2 percent. At intersections, interchange ramps or in unusual situations, the high point of the crown position may vary depending upon drainage or other controls.

For two-lane roadways, cross slope should also be adequate to provide proper drainage. The cross slope for two-lane roadways for usual conditions is 2 percent and should not be less than 1 percent.

Shoulders should be sloped sufficiently to drain surface water but not to the extent that safety concerns are created for vehicular use. The algebraic difference of cross slope between the traveled way and shoulder grades should not exceed 6 percent. Maximum shoulder slope should not exceed 10 percent. Following are recommended cross slopes for various types of shoulders:

- Bituminous and concrete-surface shoulders on tangents should be sloped from 2 to 6 percent. Often the slope rate is identical to that used on the travel lanes, for constructability, smooth transition, and ease of use during construction and maintenance traffic control.
- Gravel or crushed rock shoulders should be sloped from 4 to 6 percent.
- Turf shoulders should be sloped at about 8 percent.

For hurricane shoulder evacuation lanes (Evaculanes) and for facilities where widening is anticipated to accommodate the ultimate typical section (i.e. using the proposed shoulders as future traffic lanes) the shoulder cross-slope should be designed in accordance with the criteria for traffic lanes.

Pavement cross slopes on all roadways should not be less than 1 percent. A cross-slope should not normally exceed 3 percent on a tangent alignment unless there are three or more lanes in one direction of travel. Where 3 or more lanes are provided in one direction the cross slope should not exceed 4 percent. Cross-slopes greater than 2 percent should be limited to use in areas of intense rainfall.

Median Design

A median (i.e., the area between opposing travel lane edges) is provided primarily to separate opposing traffic streams. The general range of median width is from 4-ft to 76-ft, with design width dependent on the type and location of the highway or street facility. When determining the minimum median width, consideration should be given to the demand for U-turn movements based on local access requirements. Refer to Chapter 7, Figure 7-47 for minimum median width required to accommodate various design vehicles.

In rural areas, median sections are normally wider than in urban areas. For multi-lane rural highways without access control, a median width of 76-ft is desirable to provide complete shelter for trucks at median openings (crossovers). These wide, depressed medians are also effective in reducing headlight glare and providing a clear zone for run-off-the-road vehicle encroachments. Refer to Chapter 3 Section 5, Multi-Lane Rural Highways Section for additional information on medians in rural areas.

Where economically feasible, freeways in rural areas should also desirably include a 76-ft median. However, since freeways by design do not allow at-grade crossings, median widths do not need to be sufficient to shelter crossing trucks. In this regard, where right-of-way costs are prohibitive, reduced median widths (less than 76-ft) may be used for certain rural freeways. Statistical studies have shown that over 90 percent of median encroachments involve lateral distances traveled of 48-ft or less. To account for this, unless continuous longitudinal barriers are used, depressed medians on rural freeway sections should be 48-ft or more in width.

Urban freeways generally include narrower, flush medians with continuous longitudinal barriers. Refer to Appendix A Section 8, Median Barrier for recommendations on the use of median barriers. Medians vary in width, up to 30-ft, with 24-ft commonly used. Refer to Chapter 3 Section 6, Freeways for additional information on medians on freeways.

For low-speed urban arterial streets, flush or curbed medians are used. A width of 16-ft will effectively accommodate left-turning traffic for either raised (turn lane plus raised median) or flush medians. However, where pedestrian refuge is a consideration for raised medians, allowances for a 6-ft width raised median, measured from the back of curb, is preferred, (6-ft measured from the face of curb is the minimum requirement). Refer to Chapter 7 Section 3, Pedestrian Facilities for additional guidance. The continuous two-way left-turn lane (TWLTL) design is appropriate where a high frequency of mid-block left turns exists or are anticipated. Median types for urban arterials without access control are further discussed in Chapter 3 Section 2, Urban Streets.

When flush median designs are selected, it should be expected that some crossing and turning movements can occur in and around these medians. Full pavement structure designs will usually be carried across flush medians to allow for traffic movements.

Median encroachment countermeasures should be considered where appropriate. High severity injuries and fatalities are a result of cross median crashes on highspeed roadways. Reducing median encroachment reduces cross median crashes and fixed object crashes in the median. The following guidelines below are for reducing the frequency and severity of median related crashes on divided highways:

Design Guidance to Reduce Consequences of Median Encroachments

- Minimize potential for collision with fixed objects:
  - Relocate or remove fixed objects in median.

- Reduce consequences of collision with fixed objects:
  - Provide barrier to shield objects in median.
- Reduce likelihood of cross-median collisions:
  - Provide wider median; and
  - Provide continuous median barrier.
- Reduce likelihood of vehicle overturning:
  - Flatten median slopes;
  - Provide U-shaped (rather than V-shaped) median cross section; and
  - Provide barrier to shield steep slopes.
- Improve design of geometric elements:
  - Provide wider median;
  - Minimize sharp curves with radii less than 3,000 feet; and
  - Minimize steep grades of four percent or more.
- Improve design of mainline ramp terminals:
  - Increase separation between on-ramps and off-ramps.
- Minimize left-side exits,
  - Improve design of merge and diverge areas by lengthening speed-change lanes;
  - Simplify design of weaving areas; and
  - Increase decision sight-distance to on-ramps.

Countermeasures to Reduce Likelihood of Median Encroachments

- Reduce driver inattention:
  - Provide edgeline or shoulder rumble strips.
- Decrease side friction demand:
  - Improve/restore superelevation at horizontal curves.
- Increase pavement friction:
  - Provide high-friction pavement surfaces.
- Reduce high driver workload:
  - Improve visibility and provide better advance warning for on-ramps;
  - Improve visibility and provide better advance warning for curves and grades; and
  - Improve delineation.
- Encourage drivers to reduce speeds:
  - Provide transverse pavement markings.

- Minimize weather-related crashes:
  - Provide weather-activated speed signs;
  - Provide static signs warning of weather conditions (e.g. bridge freezes before road surface);
  - Apply sand or other materials to improve road surface friction;
  - Apply chemical de-icing or anti-icing as a location-specific treatment;
  - Improve winter maintenance response times; and
  - Raise the state of preparedness for winter maintenance.

Lane Widths

For high-speed facilities such as all freeways and most rural arterials, lane widths should be 12-ft minimum. For low-speed urban streets, 11-ft or 12-ft lanes are generally used. Subsequent sections of this manual identify appropriate lane widths for the various classes of highway and street facilities.

Shoulder Widths

Shoulders are of considerable value on high-speed facilities such as freeways and rural highways. Wide, surfaced shoulders provide a suitable, all-weather area for stopped vehicles to be clear of the travel lanes. Shoulders, in addition to serving as emergency parking, lend lateral support to travel lane pavement structure, provide a maneuvering area, increase sight distance of horizontal curves, and give drivers a sense of safe, open roadway. Design values for shoulder widths for the various classes of highways are shown in the appropriate subsequent portions of this manual.

Shoulder widths on bridge structures are measured from the nominal face of rail to the edge of traveled way. For additional guidance in reference to current standard bridge railings in Texas, reference the *TxDOT Bridge Railing Manual* and the applicable Bridge Railing Standard.

On urban collector and local streets, parking lanes may be provided instead of shoulders. On arterial streets, parking lanes decrease capacity and generally are discouraged. Subsequent sections of this manual identify appropriate shoulder widths for the various classes of highway and street facilities.

Pavement Taper Lengths

The following equations define minimum taper lengths where lanes and / or shoulders are reduced, opened, or shifted. These equations define minimum taper lengths. The project conditions (e.g., higher traffic or truck volumes) may indicate the need for additional lengths or appropriate horizontal curvature. For guidance on the length of tapers for turn lanes, acceleration lanes, or deceleration lanes, reference the respective facility type in Chapter 3.

$L = WS$, for $S \geq 45$ mph

$L = WS^2/60$, for $S < 45$ mph

Where:

L = Length of taper, ft
W = Width of offset, ft
S = Posted speed, mph

When more space is available, a longer than minimum taper distance can be beneficial.

Lane Reduction Transition Taper (L)

Lane-reduction transition tapers are used where the number of through travel lanes is reduced due to narrowing of the roadway or section of on-street parking. The minimum length of a lane reduction transition taper is *L*.

Approach Taper for Obstructions (L)

Approach tapers for obstructions are used where the width of a through travel lane is reduced because of a fixed obstruction within a paved roadway. An approach taper for an obstruction is required upstream and downstream of the obstruction. The minimum length of an approach taper for obstruction is *L*.

Lane-Opening Taper (½ L)

Lane-opening tapers are used to where a through travel lane is added without a lateral shift of the through traffic. The minimum length of a lane-opening taper is *½ L*.

Shoulder Taper (⅓ L)

Shoulder tapers are used where the improved shoulders are reduced or increased in width. The minimum length of a shoulder taper is *⅓ L*.

Shifting Taper

Shifting tapers are used to perform a lateral shift of the through traffic during temporary traffic control activities (i.e. construction, maintenance, and incident management). Shifting tapers should not be used to address permanent changes in horizontal alignment. When a change in horizontal alignment is required the design criteria from Chapter 2, Section 5 should be used. For additional information on temporary traffic control refer to the Texas MUTCD.

Curb and Curb with Gutters

Although curbs do not have a significant re-directional capacity, curbs are intended to discourage motorists from deliberately leaving the roadway. Curb designs are classified as vertical or mountable. Vertical curbs are defined as those having a vertical or nearly vertical face 6-in or higher. Mountable curbs are defined as those having a mountable face less than 6-in in height. Mountable curbs, especially those with heights of 4-in or less, can be readily traversed by a motorist when necessary. A preferable height for mountable curbs at some locations may be 4-in or less because higher curbs may drag the underside of some vehicles. Refer to the current TxDOT Curb Standard (CCCG) for illustrations on the various TxDOT standard curb types.

Curbs are used primarily on frontage roads, cross roads, and low-speed streets in urban areas. In general, curbs are not desirable along high-speed roadways. They should not be used with high-speed through traffic lanes or ramp areas except at the outer edge of the shoulder where needed for drainage, in which case they should be of the mountable type, preferably 4-in or less in height.

Roadside Design

Of particular concern in the design process is mitigating the number of single-vehicle, run-off-the-road crashes which occur even on the safest facilities. The configuration and condition of the roadside greatly affect the extent of damages and injuries for these crashes.

Increased safety may be realized through application of the following principles, particularly on high-speed facilities:

- A “forgiving” roadside should be provided, free of unyielding obstacles including landscaping, drainage facilities that create obstacles, steep slopes, utility poles, etc. For adequate safety, it is desirable to provide an unencumbered roadside recovery area that is as wide as practicable for the specific highway and traffic conditions.
- For existing highways, treatment of obstacles should be considered in the following order:
  - Remove the obstacle.
  - Redesign the obstacle so that it can be safely traversed.
  - Relocate the obstacle to a point where it is less likely to be struck.
  - Make the obstacle breakaway.
  - Apply a cost-effective device to provide for redirection (longitudinal barrier) or severity reduction (impact attenuators). Barrier should only be used if the barrier is less of an obstacle than the obstacle it would protect, or if the cost of otherwise safety treating the obstacle is prohibitive.
  - Delineate the obstacle.
- Use of higher than minimum design standards result in a driver environment which is fundamentally safer because it is more likely to compensate for driver errors. Frequently, a design,

including sight distances greater than minimum, flattened slopes, etc., costs little more over the life of a project and substantially increases safety and usefulness.

- For improved safety performance, highway geometry and traffic control devices should confirm drivers' expectations. Unexpected situations (e.g., left-side ramps on freeways, sharp horizontal curvature introduced within a series of flat curves, etc.) have demonstrated adverse effects on traffic operations.

These principles have been incorporated as appropriate into the design guidelines included herein. These principles should be examined for their applicability at an individual site based on its particular circumstances, including the aspects of social impact, environmental impact, economy, and safety.

Slopes and Ditches

Side slopes

Side slopes refer to the slopes of areas adjacent to the shoulder and located between the shoulder and the right-of-way line. For safety reasons, it is desirable to design relatively flat areas adjacent to the travel-way so that out-of-control vehicles are less likely to turn over, vault, or impact the side of a drainage channel.

Slope Rates

The path that an out-of-control vehicle follows after it leaves the traveled portion of the roadway is related to a number of factors such as driver capabilities, slope rates, and vehicular speed. Crash data indicates that approximately 75 percent of reported encroachments do not exceed a lateral distance of 30-ft from the travel lane edge where roadside slopes are 1V:6H or flatter - slope rates that afford drivers significant opportunity for recovery. Crash test data further indicates that steeper slopes (up to 1V:3H) are negotiable by drivers; however, recovery of vehicular control on these steeper slopes is less likely. Recommended clear zone width associated with these slopes are further discussed in Clear Zone.

Design Values

Particularly difficult terrain or restricted right-of-way width may require deviation from these general guide values. Where conditions are favorable, it is desirable to use flatter slopes to enhance roadside safety.

Front Slope

The slope adjacent to the shoulder is called the front slope. Ideally, the front slope should be 1V:6H or flatter, although steeper slopes are acceptable in some locations. Rates of 1V:4H or flatter facilitate efficient operation of construction and maintenance equipment. Slope rates of 1V:3H may be used in constrained conditions. Slope rates of 1V:2H are normally used on bridge header banks or ditch side slopes, both of which would likely require rip-rap. Slopes greater than 1V:2.5H require evaluation for slope stability per *TxDOT's Geotechnical Manual*.

When the front slope is steeper than 1V:3H, a longitudinal barrier may be considered to keep vehicles from traversing the slope. A longitudinal barrier should not be used solely for slope protection for rates of 1V:3H or flatter since the barrier may be more of an obstacle than the slope. Also, since recovery is less likely on 1V:3H and 1V:4H slopes, fixed objects should not be present near the toe of these slopes. Particular care should be taken in the treatment of man-made appurtenances such as culvert ends. See Appendix A, Section 2 for additional information on considerations for barrier need.

Back Slope

The back slope is typically at a slope of 1V:4H or flatter for mowing purposes. Generally, if steep front slopes are provided, the back slopes are relatively flat. Conversely, if flat front slopes are provided, the back slopes may be steeper. The slope ratio of the back slope may vary depending upon the geologic formation encountered. For example, where the roadway alignment traverses through a rock formation area, back slopes are typically much steeper and may be close to vertical. Steep back slope designs should be examined for slope stability.

Design

The intersections of slope planes in the highway cross section should be well rounded for added safety, increased stability, and improved aesthetics. Front slopes, back slopes, and ditches should be sodded and/or seeded where feasible to promote stability and reduce erosion. In arid regions, concrete or rock retards may be necessary to prevent ditch erosion.

Where guardrail is placed on side slopes, the area between the roadway and barrier should be sloped at 1V:10H or flatter.

Roadside drainage ditches should be of sufficient width and depth to handle the design run-off and should be at least 6-in below the subgrade crown to insure stability of the base course. For additional information, see Drainage Facility Placement.

Lateral Offset to Obstructions

The distance from the edge of the traveled way, beyond which a roadside object will not be perceived as an obstacle and result in a motorist's reducing speed or changing vehicle position on the roadway, is called the lateral offset. It is generally desirable that there be uniform clearance between traffic and roadside features such as bridge railings, parapets, retaining walls, and roadside barriers. In an urban environment, right of way is often limited and is characterized by sidewalks, enclosed drainage, numerous fixed objects (e.g., signs, utility poles, luminaire supports, fire hydrants, sidewalk furniture, etc.), and traffic making frequent stops.

Uniform alignment enhances highway safety by providing the driver with a certain level of expectation, thus reducing driver concern for and reaction to those objects. The lateral offset to obstructions helps to:

- Avoid impacts on vehicle lane position and encroachments into opposing or adjacent lanes;
- Improve driveway and horizontal sight distances;
- Reduce the travel lane encroachments from occasional parked and disabled vehicles;
- Improve travel lane capacity; and
- Minimize contact from vehicle mounted intrusions (e.g., large mirrors, car doors, and the overhang of turning trucks).

Where a curb is present, the lateral offset is measured from the face of curb (FOC) and must be a minimum of 1.5-ft. A minimum of 1-ft lateral offset should be provided from the toe of barrier to the edge of traveled way. A guard fence placed in the vicinity of a curb per the current TxDOT guardrail standard does not violate the minimum lateral offset requirement.

Clear Zone

A clear recovery area, or clear zone, should be provided along highways. Clear zone requirements for 4R projects are shown in Table 2-12. A clear zone is the unobstructed, traversable area provided beyond the edge of the through traveled way for the recovery of errant vehicles. The clear zone includes shoulders, bicycle lanes, and auxiliary lanes, except those auxiliary lanes that function like through lanes. Such a recovery area should be clear of unyielding objects where practical or shielded by crash cushions or barrier.

Table 2-12: 4R Clear Zones

Location	Functional Classification	Design Speed (mph)	Avg. Daily Traffic[2]	Clear Zone Width (ft)[1,3,4,5]	
-	-	-	-	Minimum	Desirable
Rural	Freeways	All	All	30 (16 for ramps)	
Rural	Arterial	All	≤ 750 ≥ 750	16 30	30 --
Rural	Collector	≥ 50	All	Use above rural arterial criteria.	
Rural	Collector	≤ 45	All	10	--
Rural	Local	All	All	10	--
Suburban	All	All	< 8,000	10[6]	10[6]
Suburban	All	All	8,000 - 12,000	10[6]	20[6]
Suburban	All	All	12,000 - 16,000	10[6]	25[6]

Table 2-12: 4R Clear Zones

Suburban	All	All	>16,000	20[6]	30[6]
Urban	Freeways	All	All	30 (16 for ramps and collector-distributor)	
Urban	All (Curbed)	≥ 50	All	Use above suburban criteria insofar as available border width permits.	
Urban	All (Curbed)[7]	≤ 45	All	4 from FOC	6 from FOC
Urban	All (Uncurbed)	≥ 50	All	Use above suburban criteria.	
Urban	All (Uncurbed)	≤ 45	All	10[6]	--

Notes:

1. Devices such as traffic signal supports, railroad signal/warning device supports, and controller cabinets must be located as far from travel lanes as feasible. If not feasible to place outside of the clear zone, these devices may be excluded from clear zone requirements. Other non-breakaway devices must be located outside the prescribed clear zone or these devices must be protected with barrier.
2. Average ADT over project life (i.e., 0.5 x (present ADT plus future ADT)). Use total ADT on two-way roadways and directional ADT on one-way roadways.
3. Without barrier or other safety treatment of appurtenances.
4. Measured from edge of travel lane for all cut sections and for all fill sections where side slopes are 1V:4H or flatter. Where fill slopes are steeper than 1V:4H it is desirable to provide a 10 ft area free of obstacles beyond the toe of slope.
5. Desirable, rather than minimum, values should be used where feasible.
6. Purchase of 5-ft or less of additional right-of-way strictly for satisfying clear zone provisions is not required.
7. For curbed facilities with a shoulder, bike lane or any buffer in addition to the curb offset, the minimum measurement begins at the edge of the through travel lane. The clear zone criteria is met if either 10-ft from the through travel lane or the distance measured from the FOC is met.

The clear zone values shown in Table 2-12 are measured from the edge of travel lane. These are appropriate design values for all cut sections (see Section 8, Drainage Facility Placement, for cross sectional design of ditches within the clear zone area) and for all fill sections with side slopes 1V:4H or flatter. It should be noted that, while a 1V:4H slope is acceptable, a 1V:6H or flatter slope is preferred for both errant vehicle performance and slope maintainability. For slopes steeper than 1V:4H, errant vehicles have a reduced chance of recovery, therefore it is preferable to provide an obstacle-free area of 10-ft beyond the toe of steep side slopes even when this area is outside the clear zone.

Section 8 — Drainage Facility Placement

Overview

This section contains information on the following topics:

- Introduction;
- Design Treatment of Cross Drainage Culvert Ends;
- Parallel Drainage Culverts; and
- Side Ditches.

Introduction

In designing drainage systems, the primary objective is to properly accommodate surface runoff along and across highway right-of-way through the application of sound hydraulic principles.

Consideration must also be given to incorporating safety into the design of drainage appurtenances. The best design would efficiently accommodate drainage and be traversable by an out-of-control vehicle without rollover or abrupt change in speed.

To meet safety needs, the designer may use one of the following treatments:

- Design or treat drainage appurtenances so that they will be traversable by a vehicle without rollover or abrupt change in speed;
- Locate appurtenances a sufficient distance, consistent with traffic volume, from the travel lanes to reduce the likelihood of collision; or
- Protect the driver through installation of traffic barrier shielding appurtenances.

The following guidelines are intended to improve roadside safety with respect to facilities accommodating drainage parallel to and crossing under highways. The guidelines apply to all rural, high-speed facilities and other facilities with posted speed limits of 50 mph or more and with rural type (uncurbed) cross sections. Where reference is made to clear zone requirements in these guidelines, see Table 2-12 and the discussions regarding Slopes and Ditches, Roadside Design, and Clear Zone in Section7. Desirable values for clear zone should generally be used and minimum clear zone widths applied where unusual conditions are encountered. Site visits may be appropriate to ascertain terrain conditions and debris potential before arriving at design decisions.

Designers should address and resolve culvert end treatment issues early in project development. If there are doubts about the proper application of criteria on a given project or group of projects, then arrangements should be made for a meeting with the appropriate entities prior to in-depth development of PS&E.

Design Treatment of Cross Drainage Culvert Ends

Cross drainage culverts are defined as those conveying drainage across and beneath the highway. Selection of an appropriate end treatment is primarily related to culvert size, culvert end location, side slope rate, terrain characteristics, drift conditions, right-of-way availability, and other considerations that may influence treatment selection at individual sites.

Roadside safety performance is related to clear zone width and side slope rate. For a discussion of safety performance and design guidelines related to side slopes, see Section 7, Slopes and Ditches Where right-of-way availability and economic conditions permit, flatter slopes may be used.

Design values for 4R clear zones are shown in Table 2-12 for new location and major reconstruction projects. Within the clear zone, side slopes should preferably be 1V:6H or flatter with 1V:4H as a maximum steepness in most cases.

Small Culverts

A small culvert is defined as a single round pipe with 36-in or less diameter, a single box culvert with span of 36-in or less, or multiple round pipes each with 30-in or less diameter, each oriented normal to the roadway. (Note: For arch pipes, use span dimension instead of diameter.)

When skews are involved, the definition of a small pipe culvert is modified as shown in Table 2-13:

Table 2-13: Maximum Diameter of Small Pipe Culvert

Skew (degree)	Single Pipe (in)	Multiple Pipe (in)
15	33	30
30	27	24
45	24	21

Small pipe culverts with sloping, open ends have been crash tested and proven to be safely traversable by vehicles for a range of speeds. Small culvert ends should be sloped at a rate of 1V:3H or flatter and should match the side slope rate, thereby providing a flush, traversable safety treatment. Single box culverts normal to the roadway with spans of 36-in or less may be effectively safety treated just as small pipes.

When vulnerable to run-off-the-road vehicles (i.e., unshielded by barrier), sloped ends should be provided on small culverts regardless of culvert end location with respect to the clear zone dimensions. For existing culverts, this often entails removing existing headwalls and may include removing the barrier treatment if it is no longer needed to protect an obstacle other than a culvert end.

For new culverts or existing culverts that may need adjusting, culvert pipe length should be controlled by the intercept of the small pipe and the side slope planes. Side slopes should not be warped or flattened near culvert locations. Also, terrain near the culvert ends should be smooth and free of fixed objects, and headwalls should not be used.

Intermediate Culverts

An intermediate size pipe culvert is defined as a single round pipe with more than 36-in diameter or multiple round pipes each with more than 30-in diameter but having maximum diameter of 60-in. For arch pipes, use span dimension instead of diameter.

Intermediate size single box culverts are defined as those having only one barrel with maximum height of 60-in. Cross sectional area of the single box or individual pipe normally should not exceed 25-ft^2.

The openings of intermediate size single barrel box and pipe culverts are too large to be safely traversed by a vehicle. Recommended safety treatment options should be considered in the following order:

1. Provide sloped ends with safety pipe runners.
2. Provide flat side slopes and locate the ends outside the clear zone.
3. Use barrier to shield culvert ends.

Sloped end treatments with safety pipe runners are preferred from a safety standpoint and are generally cost effective for both new and existing intermediate size culverts, regardless of end location with respect to clear zone criteria. These end treatments should be sloped at a rate of 1V:3H or flatter and should match the side slope rate thereby providing a flush, traversable safety treatment. Length of new culverts should be governed by the locations of the side slope plane/culvert intercepts rather than by clear zone. Terrain near the culvert end should be smoothly shaped and traversable, and headwalls should not be used.

For existing intermediate size single barrel box and pipe culverts, no treatment is necessary for culvert end offsets beyond the clear zone and below the traffic volume threshold as shown in Table 2-12. Where an improved design is warranted using Table 2-12, the removal of headwalls and installation of sloped ends with safety pipe runners is the preferred safety treatment.

In certain situations (e.g., culvert skew exceeds 15 degrees or severe debris problems) treatment with safety pipe runners may be impractical. For these conditions, locating intermediate size culvert ends to meet desirable clear zone values (see Table 2-12) is preferred over shielding with barrier. Designs having flared wing walls with safety pipe runners oriented parallel to the stream flow and spaced at 30-in maximum center to center thereby can minimize debris problems.

Large Culverts

Large multiple box culverts are defined as those with more than one barrel and a total opening (i.e., distance) of 20-ft or less between extreme inside faces as measured along the highway centerline.

Large single pipes or single box culverts are defined as those with diameter or height exceeding 5-ft or cross-sectional area exceeding 25-ft^2.

From a safety standpoint alone, treatment for both new and existing installations should be considered in the following order:

1. Provide safety pipe runners.
2. Meet or exceed desirable clear zone value.
3. Shield with barrier.

Designers should carefully consider several factors before opting to use safety pipe runners. Where a defined channel is present, it may be impossible or impractical to shape the terrain near the culvert end to provide for vehicular traversability. Such circumstances would dictate that a more suitable, culvert end treatment be selected.

Meeting clear zone criteria does not eliminate the obstacle of the culvert end, rather the obstacle is placed at a location where it is less likely to be struck. Although not as desirable as providing a traversable culvert end, it is preferred over barrier treatment where there is sufficient right-of-way and where the cost of providing the necessary culvert length is reasonable. Where the cost of added length for new culverts or of extension of existing culverts is three or more times the cost of shielding with barrier, treatment with barrier becomes an attractive alternative.

For low-volume conditions (less than 750 current ADT), the treatment option that has the lowest initial construction cost is generally the most cost-effective design if an improved design is warranted.

Bridge Class Drainage Culverts

Bridge class culverts are defined as those having an opening (i.e., distance) of more than 20-ft between the extreme inside faces as measured along the highway centerline. Bridge class culverts require protection whether they are inside or outside the clear zone. Exceptions to this requirement can be obtained by approval of a Design Exception or Design Waiver Request by the Bridge Division (see the *Bridge Railing Manual* for specific exception criteria).

Recommended treatment options are in the following priority:

1. Safety treat culvert ends.
2. Shield with appropriate barrier or attenuator. Table 2-14 provides guidelines for installing guardrails and bridge rails.

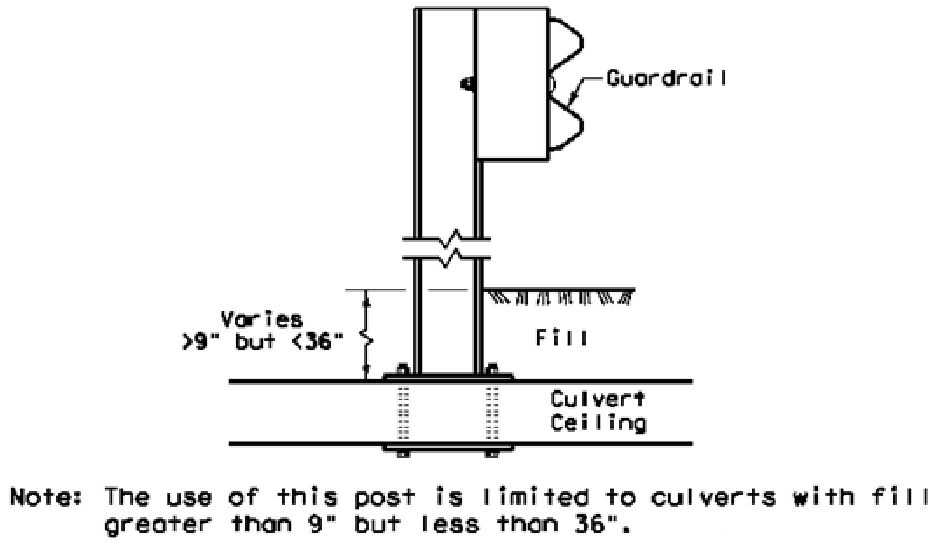
See discussion on cost comparison under Multiple Box Culverts for considerations when deciding on whether to safety treat or shield the culvert ends.

Table 2-14: Treatment Barrier Rail for Bridge Class Culverts

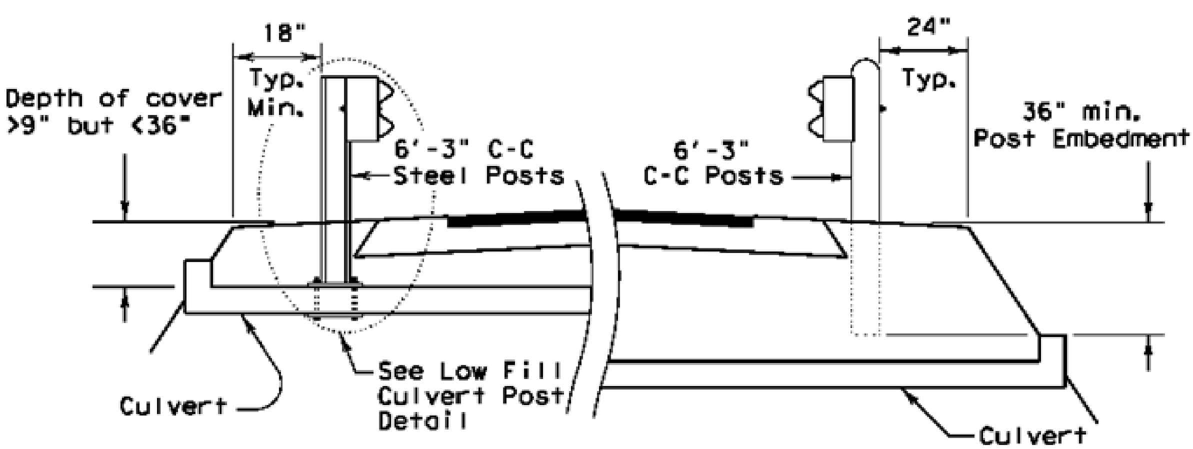
Depth of Cover (D)	Treatment
D < 9-in	Bridge Railing
9-in ≤ D < 36-in	Steel post welded to base plate and bolted to culvert ceiling (Low-fill culvert post option on Guardrail standard)
D ≥ 36-in[1]	Standard Guardrail
Note: 1. Refer to Bridge Division for further guidance.	

An additional option for shorter bridge class culverts is the use of the Long Span Guardrail when the clearance requirements on the standard are met.

Where guardrail is carried across a bridge class culvert, side slopes should be designed to provide for lateral support of the guardrail. See Figure 2-8, and the current guardrail standard for specific application.



LOW FILL CULVERT POST



USE OF GUARDRAIL AT CULVERTS (US CUSTOMARY)

Figure 2-8. Use of Guardrail at Culverts.

Parallel Drainage Culverts

The inlet and outlet points of culverts handling drainage parallel to the travel lanes, such as at driveways, side roads, and median crossovers, are concerns in providing a safe roadside environment. Flow quantities for parallel drainage situations are generally low with drainage typically accommodated by a single pipe. The following guidelines apply to driveway, side road, and median crossover drainage facilities:

- Within the clear zone, there should be no culvert headwalls or vertical ends. Outside the clear zone, single pipe ends preferably should be sloped although not required;
- Where used, sloped pipe ends should be at a rate of 1V:6H or flatter. The sloping end may be terminated and a vertical section introduced at the top and bottom of the partial pipe section as shown in Figure 2-9;
- Median crossover, side road, and driveway embankment slopes should be 1V:6H maximum steepness, with 1V:8H preferred, within the clear zone dimensions;
- Where greater than 30-in diameter pipe ends are located within the clear zone, safety pipe runners should be provided with a maximum slope steepness of 1V:6H with 1V:8H preferred. Typical details for a driveway, side road, or median crossover grate are shown in Figure 2-10. Cross pipes are not required on single, small (30-in or less diameter) pipes regardless of end location with respect to clear zone requirements; however, the ends of small pipes should be sloped as described above and appropriate measures taken to control erosion and stabilize the pipe end. Multiple 30 in. pipes require cross pipes;
- The use of paved dips, instead of pipes, is encouraged particularly at infrequently used driveways such as those serving unimproved private property; and
- For unusual situations, such as driveways on high fills or where multiple pipes or box culverts are necessary to accommodate side or median ditch drainage, the designer should consider the alternatives available and select an appropriate design.

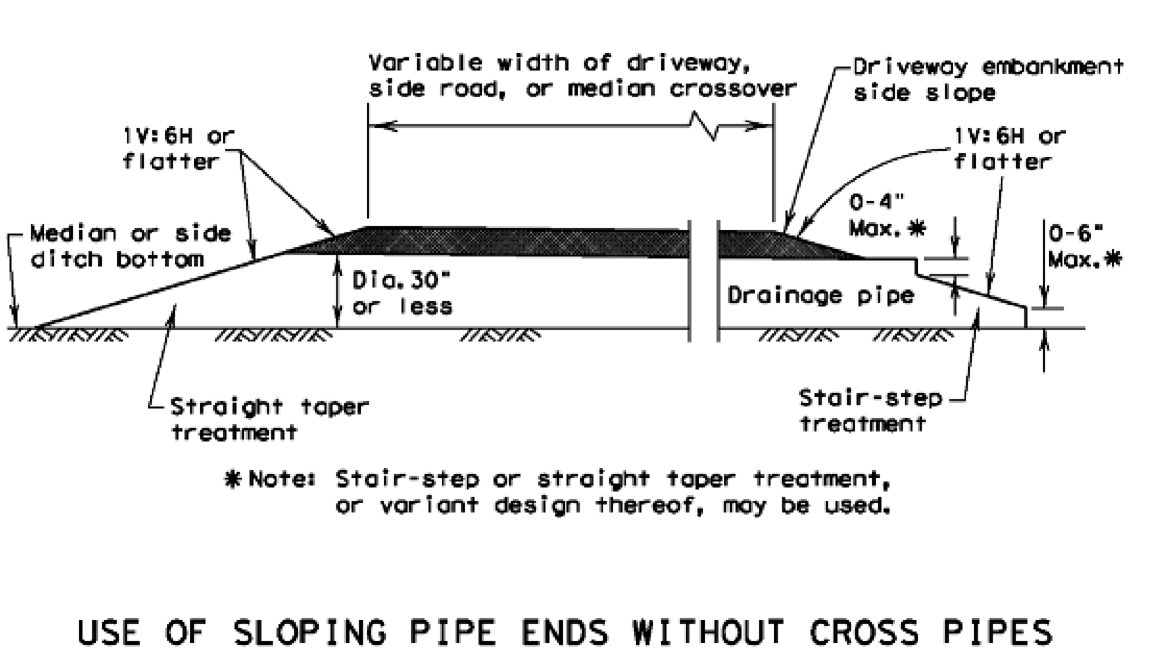


Figure 2-9. Use of Sloping Pipe Ends without Cross Pipes.

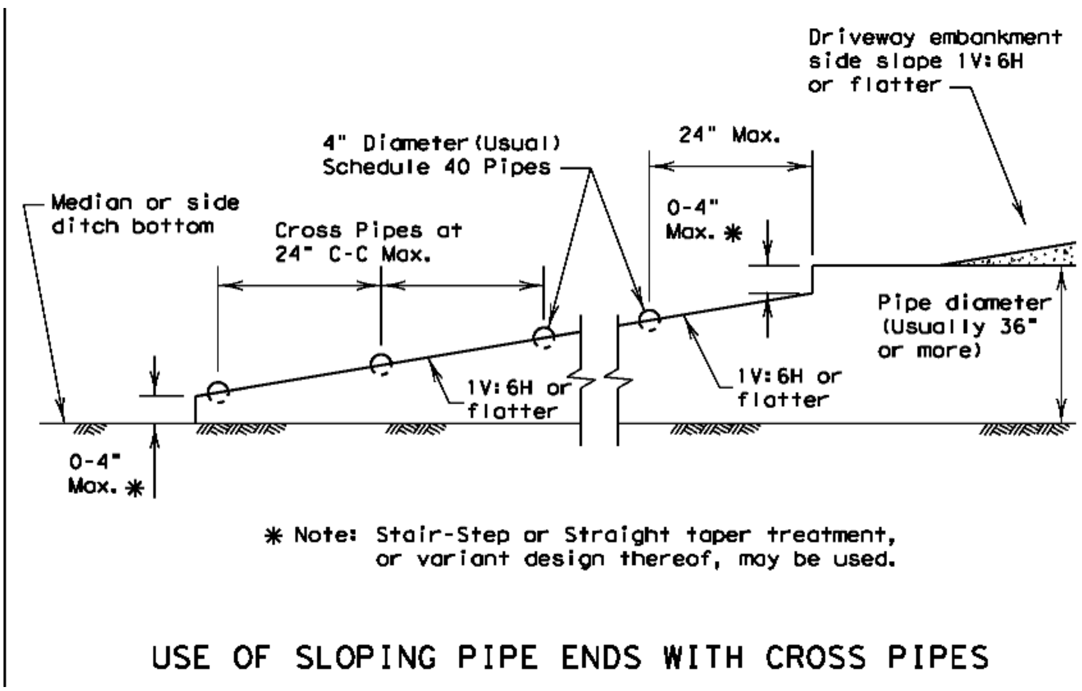


Figure 2-10. Use of Sloping Pipe Ends with Cross Pipes.

Side Ditches

For side ditches, attention to cross section design can reduce the likelihood of serious injuries during vehicular encroachments. Ditches with the cross sectional characteristics defined in Table 2-15 are preferred and should especially be sought when ditch location is within the clear zone requirements. Where conditions dictate, such as insufficient existing right-of-way to accommodate the desired ditch cross section or where ditches are located outside the clear zone requirements, other ditch configurations may be used. Typically, guardrail is not necessary where the desired ditch cross sections are provided. For additional information on general applications of roadside barriers, see Appendix A, Section 2.

Table 2-15: Desirable Ditch Cross Sections

Given Front Slope (Vertical:Horizontal)	Desirable Maximum Back Slope (Vertical:Horizontal)	
	V-Shaped	**Trapezoidal-Shaped[1]**
1V:8H	1V:3.5H	1V:2.5H
1V:6H	1V:4H	1V:3H
1V:4H	1V:6H	1V:4H
1V:3H	Level	1V:8H
Notes: 1. Trapezoidal channel bottom widths equal to or greater than 4-ft.		

Ditches that include retards to control erosion should be avoided inside the clear zone requirements and should be located as far from the travel lanes as practical unless the retardant is a rock filter dam with side slopes of 1V:6H or flatter. Non-traversable catch or stilling basins should also be located outside the clear zone requirements.

Section 9 — Roadways Intersecting Department Projects

Off-system roadways that intersect or tie into a facility which the Department is constructing should desirably meet or exceed the Department's design criteria based on the functional classification of the intersecting roadway as designated by the Transportation Planning and Programming (TPP) Division. At a minimum, the project must improve or retain the existing geometry of the intersecting roadway. There is no need for a design exception or design waiver if the off-system intersecting road's geometry retains or exceeds its existing geometry. Existing geometry includes horizontal alignment, vertical alignment, and cross-sectional elements as outlined in Sections 4, 5, and 6 of this chapter. The definition of off-system intersecting roadways excludes private driveways.

When an off-system intersecting road is modified at the request of the owner, then it must follow the Department's design criteria for the functional class of the road or street being modified. If the Department's design criteria cannot be met, then the corresponding design exception(s) and or design waiver(s) is required.

Chapter 3 — New Location and Reconstruction (4R) Design Criteria

Contents:

Section 1 — Overview

Introduction

This chapter presents guidelines applicable to all new location and reconstruction (4R) projects for several different roadway facilities including:

- Urban Streets;
- Suburban Roadways;
- Two-lane Rural Highways;
- Multilane Rural Highways;
- Freeways; and
- Texas Highway Freight Network (THFN).

For the purposes of this chapter, 4R is defined as projects that either provide a new roadway or reconstruction to upgrade an existing roadway to meet geometric criteria for a new facility. In addition to work described under resurfacing, restoration and rehabilitation, reconstruction work generally includes substantial changes in the geometric character of the highway, such as widening to provide additional through lanes, significant horizontal or vertical realignments, major improvements to the pavement structure to improve long term service, or bridge replacement. Further discussion of this definition can be found in AASHTO's *A Policy of Geometric Design of Highways and Streets*.

Note that additional vertical clearance requirements may apply to projects on the Texas Highway Freight Network (THFN) if the project criteria as specified in Chapter 3, Section 8 are met.

Departures from these guidelines are governed in Chapter 1, Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight Network (THFN) Design Deviations, Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight Network (THFN) Design Deviations.

Section 2 — Urban Streets

Overview

"Urban Streets," as used in this chapter refers to roadways in developed areas that provide access to abutting property as well as movement of vehicular traffic. Access to these facilities is controlled through driveway locations, medians, and intersections with other roadways. The TxDOT Statewide Planning Map provides guidance on area types.

Level of Service

Urban streets and their auxiliary facilities should be designed for Level of Service (LOS) B as defined in the *Highway Capacity Manual*. Densely developed urban areas may necessitate the use of LOS D. The functional class of urban facility according to the Statewide Planning Map should be used to determine the appropriate LOS. For more information regarding LOS as it relates to facility design, see Service Flow Rate under subheading Traffic Volume in Chapter 2, Section 3.

Basic Design Features

This subsection includes information on the following basic design features for urban streets:

- Geometric Design Criteria for Urban Streets;
- Medians;
- Median Openings;
- Clear Zones;
- Borders;
- Berms;
- Grade Separations and Interchanges;
- Right-of-Way Width;
- Intersections;
- Speed Change Lanes;
- Horizontal Offsets; and
- Bus Facilities.

Table 3-1 shows tabulated basic geometric design criteria for urban arterial, collector, and local streets. The basic design criteria shown in this table reflect minmum and desirable values applicable to new location, reconstruction or major improvement projects. See Chapter 2 for additional guidance on choosing an appropriate design speed.

Table 3-1: Geometric Design Criteria for Urban Streets

Item	Functional Class	Desirable	Minimum
Design Speed (mph)	All	Up to 60	30
Horizontal Radius	All	See Table 2-3, Table 2-4, and Table 2-5	
Maximum Grade (%)	All	See Table 2-9	
Stopping Sight Distance	All	See Table 2-1 and Figure 2-3	
Width of Travel Lanes (ft)	Arterial Collector Local	12 12 12	11[1] 11[2] 11[2,3]
Curb Parking Lane Width (ft)	Arterial Collector Local	12 10 9	10[4] 8[5] 8[5]
Shoulder Width (ft), Uncurbed Urban Streets[12, 13]	Arterial Collector Local	10 8 8	4[6] 3[6] 2[6]
Width of Speed Change Lanes (ft)	Arterial and Collector Local	11-12[7] 10-12[7]	10 9
Offset to Face of Curb (ft)	All	2	1
Median Width	All	See Medians	
Border Width (ft)	Arterial and Collector	20	15
	Local	10[8]	
Right-of-Way Width	All	Variable [9]	
Clear Sidewalk Width (ft)[11]	All	6-8[10]	5
On-Street Bicycle Lane Width	All	See Chapter 6, Bicycle Facilities	
Superelevation	All	See Chapter 2, Superelevation Rate, Superelevation Transition Length, Superelevation Transition Placement,Superelevation Transition Type	
Vertical Clearance at New Structures	All	See Table 2-11	
Clear Zone Width	All	See Table 2-12	
Turning Radii	All	See Chapter 7, Minimum Designs for Truck and Bus Turns	

Table 3-1: Geometric Design Criteria for Urban Streets

Item	Functional Class	Desirable	Minimum

Notes:
1. In highly restricted locations or locations with less than 5% trucks and speeds less than or equal to 40 mph, 10-ft permissible. Engineering judgement should be exercised when determining if 10-ft is acceptable and must be approved by the district.
2. In industrial areas 12-ft usual, and 11-ft minimum for restricted ROW conditions. In non-industrial areas, 10-ft minimum.
3. In residential areas, 9-ft minimum.
4. Where there is no demand for use as a future through lane, 8-ft minimum.
5. In non-commercial and non-industrial areas, 7-ft minimum.
6. Where only minimum width is provided, it should be fully surfaced. Where desirable width is provided, partial (not less than minimum width) surfacing or full width surfacing may be provided at the option of the designer.
7. See AASHTO's *A Policy on Geometric Design of Highways and Streets* for further discussion on appropriate lane width for different conditions.
8. Where available right-of-way is limited and in areas of high right-of-way costs, an absolute minimum border width of 2-ft may be acceptable where there is no sidewalk and adequate sight distance is provided.
9. Right-of-way width is a function of roadway elements as well as local conditions.
10. Wider than 6-ft is applicable for commercial areas, school routes, or other areas with concentrated pedestrian traffic.
11. Cross slopes, ramps, and sidewalks must be in compliance with the Americans with Disabilities Act Accessibility Guidelines and the Texas Accessibility Standards. See Chapter 7, Section 3, Curb and Curb and Gutters and Sidewalks and Pedestrian Facilities for additional information.
12. A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated. Off-system Bridges, with current ADT greater than 400 ADT, where this addition may represent an unreasonable increase in cost may be excepted from the bicycle clear space requirement. See Ch. 6 Section 1 for specific off-system bridge requirements for current ADT of 400 or less.
13. Where right turn lanes are present on uncurbed facilities, a 4-ft fully surfaced shoulder must be provided.

For minor rehabilitation projects where no additional lanes are proposed, existing curbed cross sections should be compared with the design criteria in Table 3-1 to determine the practicality and economic feasibility of minor widening to meet the prescribed standards. Where only minimal widening is required to conform with a standard design, it is often cost effective to retain the existing street section, thereby sparing the cost of removing and replacing concrete curb and gutter and curb inlets. For these type projects, Resurfacing, Restoration, and Rehabilitation (3R) guidelines are usually applicable, see Chapter 4.

Medians

Medians are desirable for urban streets with four or more traffic lanes. The primary functions of medians are to provide the following:

- Storage space for left-turning vehicles;
- Separation of opposing traffic streams; and
- Access control to/from minor access drives and intersections.

Medians used on urban streets include the following types:

- Raised;
- Flush; and
- Two-way left-turn lanes.

Raised Medians. A raised median is used on urban streets where it is desirable to control or restrict mid-block left-turns and crossing maneuvers. Installing a raised median can result in the following benefits:

- Restricting left-turn and crossing maneuvers to specific locations or certain movements;
- Improving traffic safety;
- Increasing throughput capacity and reducing delays; and
- Providing pedestrian refuge areas.

A raised median design should be considered where:

- ADT exceeds 20,000 vehicles per day;
- New development is occurring, and volumes are anticipated to exceed 20,000 vehicles per day; or
- There are operational concerns for mid-block turns.

For these conditions, a raised median may improve safety by separating traffic flows and controlling left-turn and crossing maneuvers. The use of raised medians should be discouraged where the roadway cross-section is too narrow for U-turns.

For median left turn lanes at intersections, a median divider width of 4-ft (measured to face of curb) is recommended to accommodate a single left turn lane. Where a pedestrian refuge is needed, a median divider width of 6-ft is required (measured minimum to face of curb, desirable to back of curb). See Chapter 7, Pedestrian Facilities for additional guidance. To prevent recurring damage to the divider, the divider should be at least 2-ft wide (measured to face of curb). If pedestrians are expected to cross the divider, then the divider should be a minimum of 5-ft wide (normal to direction of pedestrian travel) to accommodate a cut-through landing or refuge area that is at least 5-ft x 6-ft. See Dual Left-Turn Lanes for additional median width discussion.

See Appendix E, Median U-Turn Intersection (MUT) for information on alternate intersection design for roadways with raised medians.

Flush Medians. Flush medians are medians that can be traversed. Although a flush median does not permit left-turn and cross maneuvers, it cannot physically prevent these maneuvers because the median can be easily crossed. Therefore, for urban arterials where access control is desirable, flush medians should not be used.

A flush median design should include the following:

- Delineation from through lanes using double yellow stripes and possibly a contrasting surface texture or color to provide visibility; and
- Flexibility to allow additional left-turn bay storage if necessary.

Two-Way Left-Turn Lanes. Two-way left-turn lanes (TWLTL) are flush medians that may be used for left turns by traffic from either direction on the street. The TWLTL is appropriate where there are operational concerns for mid-block turns, such as areas with (or expected to experience) moderate or intense strip development. Used appropriately, the TWLTL design can improve the safety and operational characteristics of streets as demonstrated through reduced travel times and crash rates.

Recommended median lane widths for the TWLTL design are as shown in Table 3-2. When applying these criteria to new location projects or on reconstruction projects where widening necessitates the removal of exterior curbs, the median lane width should not be less than 12-ft, and preferably the corresponding desirable value shown in Table 3-2. Minimum values shown in Table 3-2 are appropriate for restrictive right-of-way projects and improvement projects where attaining the desirable width would necessitate removal and replacement of exterior curbing to gain a small amount of roadway width.

Table 3-2: Median Lane Widths for Two-Way Left-Turn Lanes

Design Speed (mph)	Width of TWLTL (ft)	
	Desirable[1]	Minimum
≤ 40	14	11
45 or 50	14	12
> 50	16	14
Note: 1. Maximum width of TWLTL should not exceed 16-ft to avoid driver confusion		

A site can be considered suitable for the use of a TWLTL when an urban street meets the following criteria:

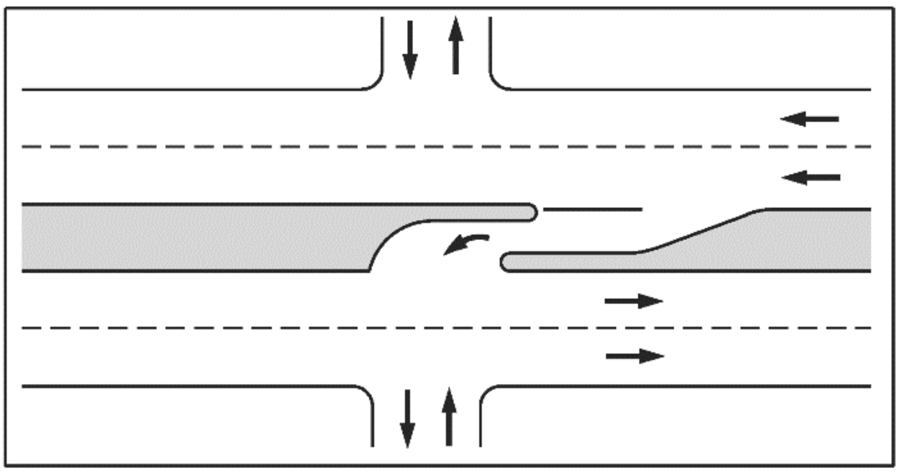
- Future ADT volume of greater than 3,000 vehicles per day for an existing two-lane urban street, 6,000 vehicles per day for an existing four-lane urban street, or 10,000 vehicles per day for an existing six-lane urban street; and
- Side roads plus driveway density of 20 or more entrances per mile.

When the above two conditions are met, the site should be considered suitable for the use of a TWLTL.

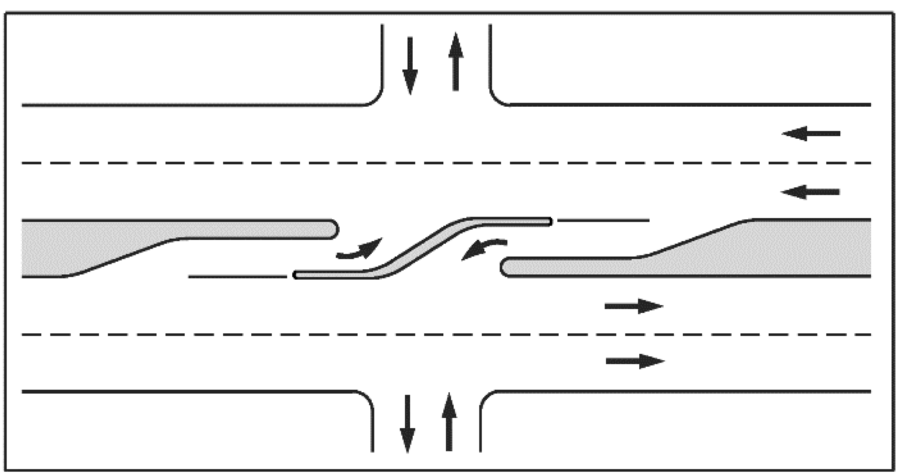
All cross sections should be evaluated for pedestrian crossing capabilities. See Chapter 7, Pedestrian Facilities for additional guidance.

Median Openings

Median openings should only be provided for street intersections or at intervals for major developed areas. Spacing between median openings must be adequate to allow for introduction of left-turn lanes and to prevent false calls at signal detection loops. Directional openings, pictured in Figure 3-1, can be used to limit the number and type of conflicts.



Left-Turn Ingress from One Direction



Left-Turn Ingress from Two Directions

Figure 3-1. Types of Directional Openings.

The positive offset design shown in Figure 3-2 (c) can improve the sight distance for left-turning vehicles where there are opposing left-turn lanes. This design has also been found to substantially reduce the frequency of left-turn crashes compared to negative (a) or no offset (b) designs and is desirable for use where practical. Further discussion and examples of offset left-turn lanes can be found in AASHTO's *Policy on Geometric Design of Highways and Streets*.

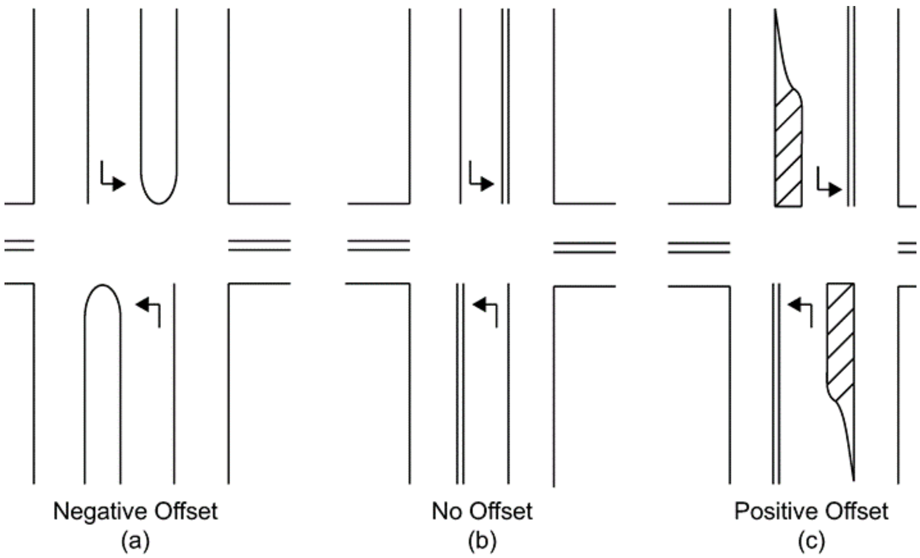


Figure 3-2. Examples of Left-Turn Lanes with Negative, Zero, and Positive Offset. Source: AASHTO's A Policy on Geometric Design of Highways and Streets

An important factor in designing median openings is the shape of the median end or median nose. The median end shape can directly alter the effective turning path the design vehicle can make. The shape of a median nose should be designed to accommodate the turning path of the design vehicle. One form of a median end at an opening is a semicircle, which is a simple design that is satisfactory for median widths less than 10-ft wide. One alternate median end design that fits the paths of design vehicles is a bullet nose. The bullet nose is formed by two symmetrical arcs with a small radius to round the nose. Consider the use of local design standards for median noses where available. In the absence of a local standard refer to Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets* for more information regarding the design of median openings.

Clear Zones

Table 2-12 presents the general clear zone guidelines for urban roadways.

Borders

The border is the area between the roadway and right-of-way line that accommodates sidewalks, provides sight distance, and utility accommodation, and separates traffic from privately owned areas. Every effort should be made to provide wide borders to serve functional needs, reduce traffic nuisances to adjacent development, and for aesthetics. Minimum and desirable border widths are as listed in Table 3-1.

Berms

There are two different types of berms typically used on urban streets. The first, constructed as a narrow shelf or path, is typically used to provide a flush grade behind a curb to accommodate the possible future installation of sidewalks. The width of the berm should accommodate a buffer between the curb and the sidewalk, the sidewalk, and any needed buffer on the backside of the sidewalk. The second type of berm is constructed as a raised mound to facilitate drainage or for landscaping purposes. A raised mound berm should be placed outside of the clear zone, when practical. If it cannot be placed outside of the clear zone, care should be taken to ensure that the slopes and configurations of the berm meet the clear zone requirements as discussed in Slopes and Ditches in Chapter 2.

Grade Separations and Interchanges

Although uncommon on urban streets, grade separations and interchanges may sometimes be the only means available for providing sufficient capacity at critical intersections. However, single or multiple grade separations do not usually improve the throughput of a roadway when other intersections are signalized along the street. With the exception of grade separations at railroads, grade separations on urban streets are usually diamond interchanges with four legs. Locations considered include high-volume intersections and where terrain conditions favor separation of grades. See Appendix E and FHWA's *Intersection Control Evaluation* (ICE) for additional information on optimal geometric configurations and traffic control solutions for interchanges.

Where feasible the width of the entire roadway approach, including parking lanes or shoulders, should be carried across or under the separation. Interchange design elements may have slightly lower dimensional values as compared to freeways due to the lower speeds involved. For example, the length of diamond ramps may be controlled by the minimum distance to overcome the elevation difference at suitable gradients.

In some instances, it may be feasible to provide grade separations or interchanges at all major crossings for a long section of arterial street. The urban street then assumes the operating characteristics and appearance of a freeway. In these instances, it may be appropriate to eliminate the remaining at-grade crossings and control access by design (i.e., provide continuous frontage roads) where right-of-way availability permits. It is not desirable to intermix facility types by providing intermittent sections of fully controlled and non-controlled access facilities.

For additional discussion on grade separations and interchanges, see Grade Separations and Interchanges and Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*.

Right-of-Way Width

Right-of-Way width is the area necessary to accommodate the various cross-sectional elements, including widths of travel and turning lanes, bicycle lanes, shoulders or parking lanes, median, borders, sidewalks, sidewalk offsets, slopes and provide ramps or connecting roadways where interchanges are involved.

The width of right-of-way for urban streets is influenced by the following factors:

- Traffic volume requirements;
- Land use;
- Availability and cost; and
- Extent of expansion.

Intersections

The number, design, and spacing of intersections influence an urban street's capacity, speed, and safety. Capacity analysis of signalized intersections is one of the most important considerations in intersection design. Traffic volumes, operational characteristics, and traffic control measures closely influence the dimensional layout or geometric design considerations of intersections.

Space limitations and lower operating speeds on urban streets reduce the curb or edge of pavement radii for turning movements as compared to rural highway intersections. Smaller radii are also advantageous for pedestrian safety as they may encourage reduced speeds for turning movements. Effective turning radii (i.e. edge line pavement markings) of 15-ft to 25-ft permit passenger cars to negotiate right turns with little to no encroachment on other lanes. Where heavy volumes of trucks or buses are present, increased effective turning radii of 30-ft to 50-ft may expedite turns to and from through lanes. To optimize the actual curb or edge of pavement radii for combination tractor-trailer units and buses, refer to Minimum Designs for Truck and Bus Turns, Chapter 7.

In general, intersection design should be simple and free of complicated channelization to minimize driver confusion. Sight distance is an important consideration even in the design of signalized intersections since, during the low volume hours, flashing operation may be used (see discussion in Intersection Sight Distance, Chapter 2). For information on the design of Alternative Intersections, reference Appendix E.

Figure 3-3 illustrates lines of sight for a vehicle entering an intersection.

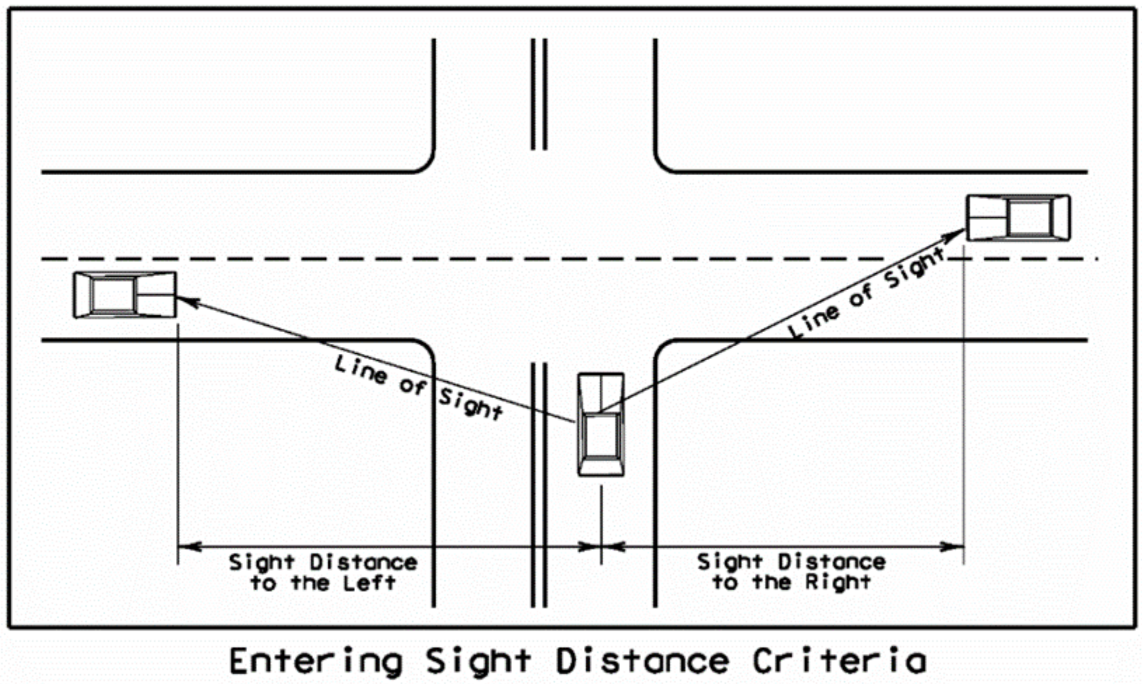


Figure 3-3. Entering Intersection Lines of Sight.

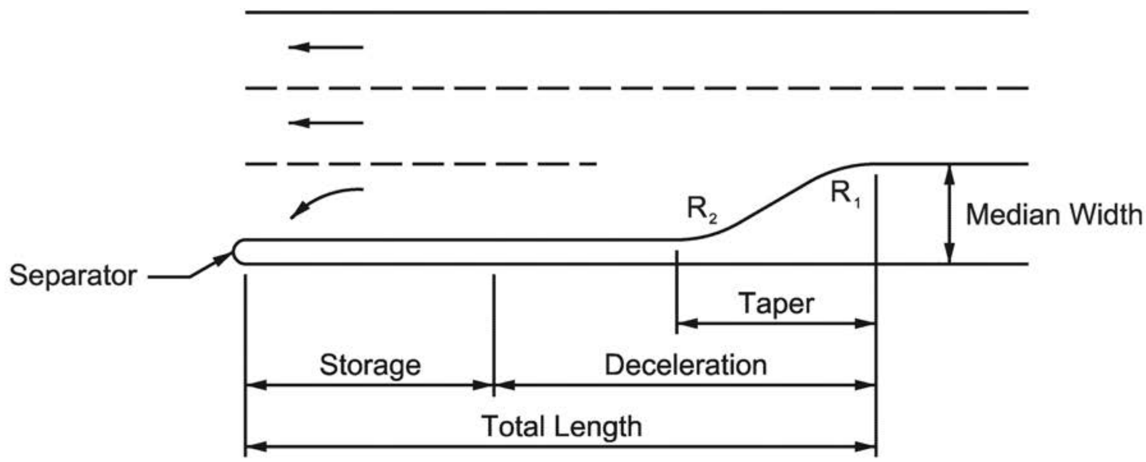
Speed Change Lanes

Speed Change Lanes are defined as acceleration or deceleration lanes for left or right turns, exit or entrance acceleration or deceleration lanes, or climbing lanes. A design waiver is required for speed change lanes that do not meet minimum length and width criteria.

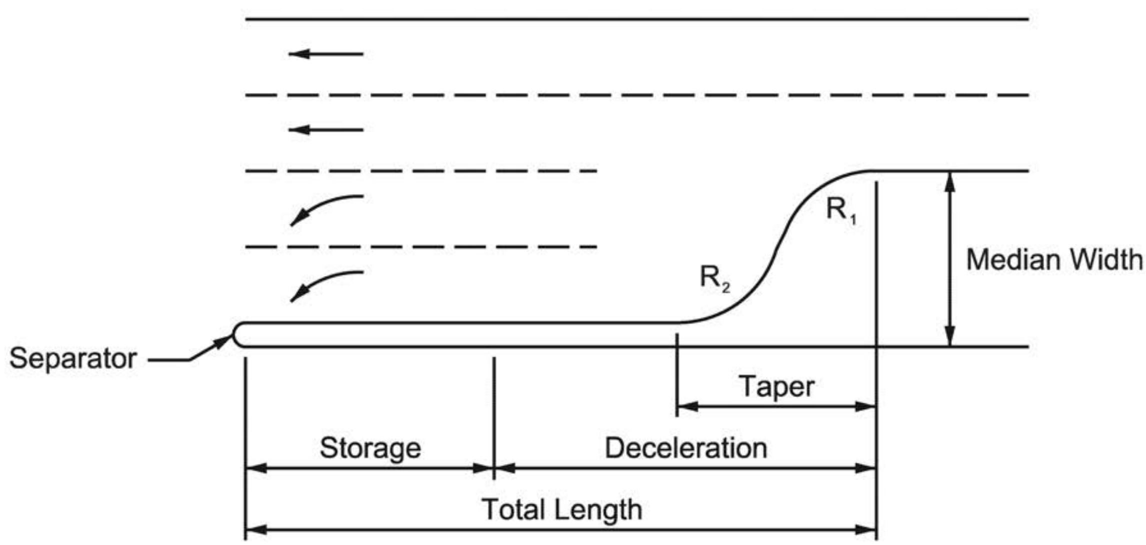
On urban streets, speed change lanes generally provide space for the deceleration and optional storage of turning vehicles. The length of speed change lanes for turning vehicles consists of the following two components:

- Deceleration length; and
- Storage length.

Left-Turn Deceleration Lanes. Figure 3-4 illustrates the use of left-turn lanes on urban streets. A short symmetrical reverse curve taper or straight taper may be used. For median left-turn lanes at intersections, a median divider width of 4-ft (measured to face of curb) is recommended. Where pedestrian refuge is needed, a median divider width of 6-ft is required (measured minimum to face of curb, desirable to back of curb). If pedestrians are expected to cross the divider, then the divider should be a minimum of 5-ft wide (normal to direction of pedestrian travel) to accommodate a cut-through landing or refuge area that is at least 5-ft x 6-ft. For illustrations of these two cases, see Chapter 7 Pedestrian Facilities for additional guidance.



(See Table 3-3 for taper, deceleration and storage lengths)
SINGLE LEFT-TURN LANE



(See Table 3-4 for taper, deceleration and storage lengths)
DUAL LEFT-TURN LANE

Figure 3-4. Left-Turn Lanes on Urban Streets.

Table 3-3 provides recommended taper lengths, deceleration lengths, and storage lengths for left-turn lanes. These guidelines may also be applied to the design of right-turn lanes.

Table 3-3: Lengths of Single Left-Turn and Right-Turn Lanes on Urban Streets [1]

Design Speed (mph)	Deceleration Length[2] (ft) Speed Differential[3]			Taper Length (ft)	Minimum[6, 7] Storage Length (ft)
	None	5-mph[4]	10-mph[5]		
30	150	105	70	50	100
35	205	150	105	50	100
40	265	205	150	50	100
45	340	265	205	100	100
50	415	340	265	100	100
55	505	415	340	100	100
60	600	505	415	100	100

Notes:

1. The minimum length of a left-turn lane is the sum of the deceleration length plus queue storage. In order to determine the design length, the deceleration plus storage length must be calculated for peak and off-peak periods, the longest of these two lengths will be the minimum design length.
2. Based on 6.5 ft/s2 deceleration to stopped condition throughout the entire length. Larger deceleration rates may be used when deceleration lengths based on 6.5 ft/s2 are impractical.
3. Speed differential = the difference between the assumed speed of a turning vehicle at the moment when it arrives at the taper and the design speed of the roadway.
4. Based on 6.5 ft/s2 deceleration from 5 mph less than design speed to stopped condition throughout the entire length.
5. Based on 6.5 ft/s2 deceleration from 10 mph less than design speed to stopped condition throughout the entire length.
6. See Storage Length Calculations discussion. For right-turn lanes the minimum queue storage is 30-ft.
7. The minimum storage length applies when: (1) the required queue storage length calculated is less than the minimum length, or (2) there is no rational method for estimating the left-turn volume. A design waiver will be required only if the 100 ft minimum storage length cannot be provided.

Deceleration Length. Deceleration length, with no speed differential, as shown in Table 3-3 assumes that deceleration starts at the beginning of the taper and continues to a stopped condition. Where providing this deceleration length is impractical, it may be acceptable to assume that turning vehicles will begin decelerating prior to arriving at the taper and clearing the through traffic lane. Using this assumption, see Table 3-3 for 5 mph and 10 mph speed differential deceleration lengths.

Storage Length Calculations. The required storage may be obtained using an acceptable traffic model such as the latest version of the *Highway Capacity Manual* software (HCS), SYNCHRO, or VISSIM or other acceptable simulation models. Where such model results have not been applied, the following formulas may be used:

Signalized:

$$L = \left(\frac{V}{N}\right)(2)(S)$$

Where:

- *L* = storage length, ft
- *V* = left-turn volume per hour, vph
  Consider multiple turn lanes when V >150.
- *N* = number of cycles per hour
  Recommend between 20 and 25 cycles per hour for peak period operations if unknown.
- 2 = a factor that provides for storage of all left-turning vehicles on most cycles
  A value of 1.8 may be acceptable on collector streets.
- *S* = queue storage length, in feet, per vehicle

% trucks	S (ft)
<5	25
5-9	30
10-14	35
15-19	40

Unsignalized:

$$L = \left(\frac{V}{30}\right)(2)(S)$$

Where:

- *L* = storage length in feet
- *V* = left-turn volume per hour, vph
- 2 = a factor that provides for storage of all left-turning vehicles on most cycles
  A value of 1.8 may be acceptable on collector streets.
- *S* = queue storage length, in feet, per vehicle

% trucks	S (ft)
<5	25

% trucks	S (ft)
5-9	30
10-14	35
15-19	40

Dual Left-Turn Deceleration Lanes. For major signalized intersections where high peak hour left-turn volumes exceeding 150 vehicles per hour are expected, dual left-turn lanes should be considered. As with single left-turn lanes, dual left-turn lanes should include lengths for deceleration, storage, and taper. Table 3-4 provides recommended lengths for dual left-turn lanes.

Table 3-4: Lengths of Dual Left-Turn and Right-Turn Lanes on Urban Streets [1]

Design Speed (mph)	Deceleration Lengths[2](ft)	Taper Length (ft)	Minimum[3, 4] Storage Length (ft)
30	150	100	100
35	205	100	100
40	265	100	100
45	340	150	100
50	415	150	100
55	505	150	100
60	600	150	100

Notes:
1. The minimum length of a left-turn lane is the sum of the deceleration length plus queue storage. In order to determine the design length, the deceleration plus storage length must be calculated for peak and off-peak periods, the longest total length will be the minimum design length.
2. Based on 6.5 ft/s^2 deceleration to stopped condition throughout the entire length. Larger deceleration rates may be used when deceleration lengths based on 6.5 ft./s^2 are impractical.
3. See Storage Length Calculations discussion.
4. The minimum storage length shall apply when: (1) the required queue storage length calculated is less than the minimum length, or (2) there is no rational method for estimating the left-turn volume. A design waiver will be required only if the 100 ft minimum storage length cannot be provided.

Right-Turn Deceleration Lanes. Figure 3-5 illustrates a right-turn deceleration lane. The length of a single right-turn deceleration lane is the same as that for a single left-turn lane (Table 3-3). However, the minimum queue storage is 30 ft for right-turn lanes. The length for a dual right-turn lane is the same for a dual left-turn lane (Table 3-4). Refer to the TxDOT Access Management *Manual* for guidelines as to when to consider a right-turn deceleration lane.

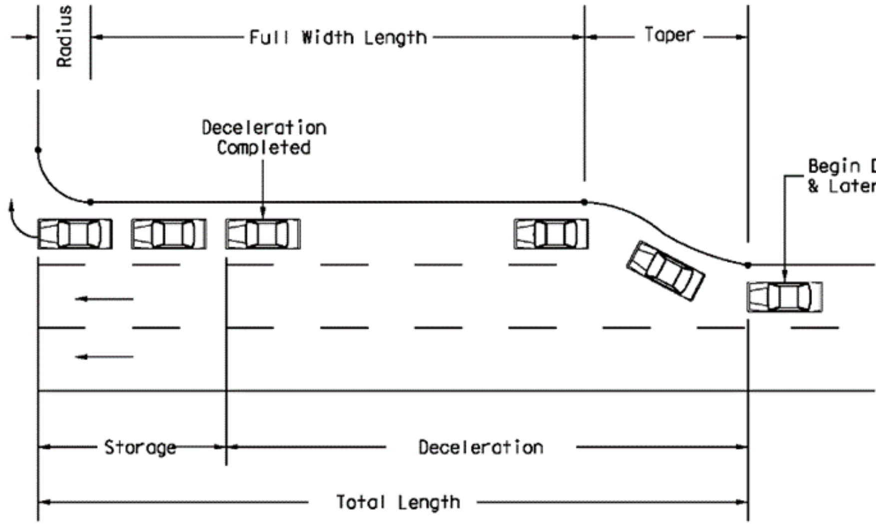
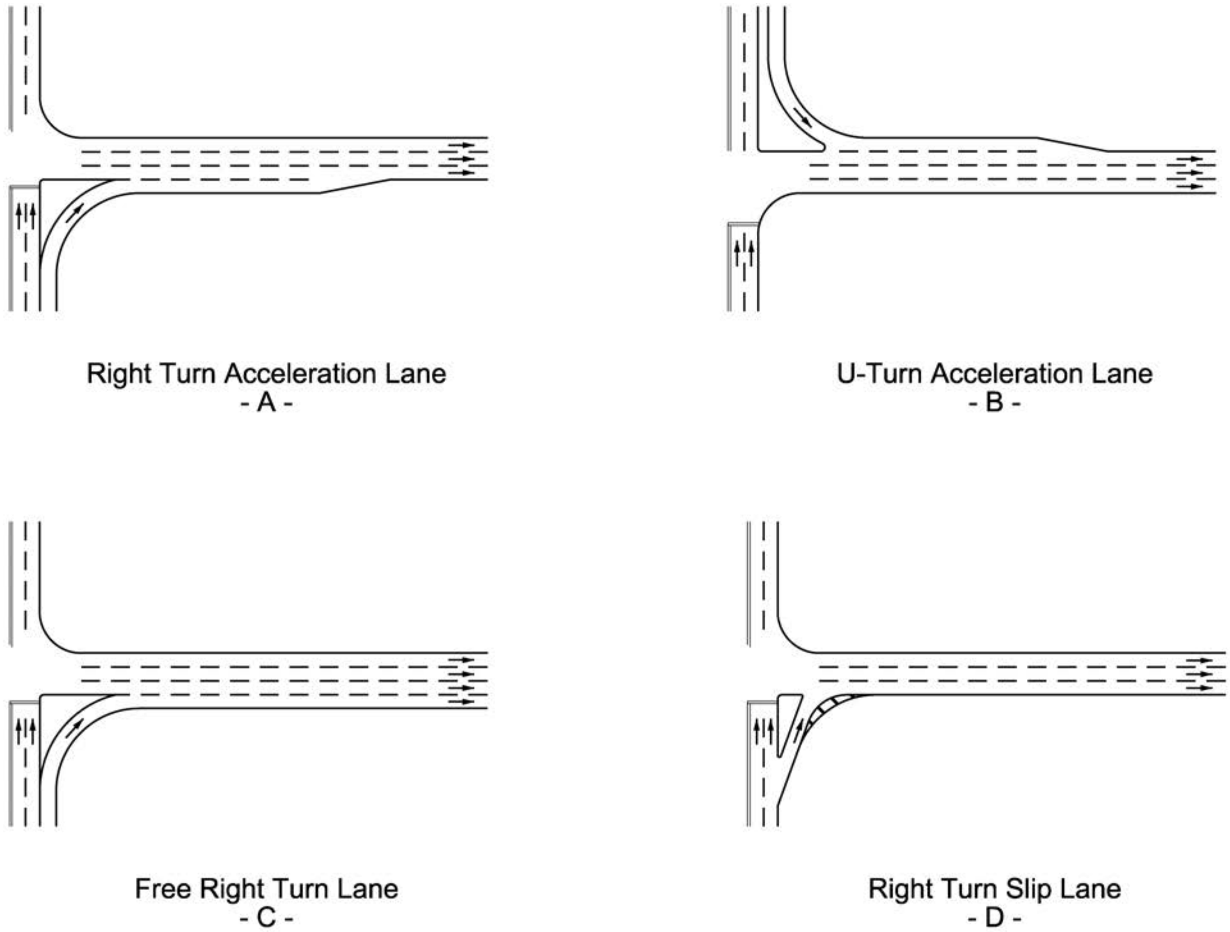


Figure 3-5. Lengths of Right-Turn Deceleration Lanes.

Right-Turn and U-Turn Acceleration Lanes. Acceleration lanes are generally not used on urban streets (See Figure 3-6(A)). See Table 3-13 for acceleration distances and taper lengths if an acceleration lane is necessary. Right turn lanes into acceleration lanes that end must be yield controlled. This also applies to U-turn lanes at grade separated interchanges (See Figure 3-6(B)).

Free Right-Turn Lanes. When an acceleration lane does not end, it can be considered a free right turn lane where stop or yield control may not be provided (See Figure 3-6(C)). It is recommended not to have down-stream driveways where free right turn lanes are provided. Refer to the TxDOT *Access Management Manual* for driveway spacing requirements.



Notes:

1. This is not intended to show striping or pavement marking details. Refer to the Texas MUTCD for information on striping and pavement marking details.
2. Refer to Appendix D for additional information on Right Turn Slip Lanes.

Figure 3-6. Types of Right Turn Treatments at Intersections.

Drainage Horizontal Offsets

For low-speed streets, cross drainage culvert ends should be offset at least 4-ft from the back of the curb or 4-ft from the outside edge of the shoulder. The designer should make the best use of the available border width to obtain wide clearances. Sloped open ends may be used effectively as safety treatments for small culverts. Consideration should be given to future sidewalk needs.

Bus Facilities

Urban areas benefit from the effective bus utilization of downtown and radial arterial streets and the effective coordination of transit and traffic improvements. To maintain and increase bus patronage, bus priority treatments on arterial streets may be used to underscore the importance of transit use. Possible bus priority treatments on non-controlled access facilities include measures designed to separate car and bus movements and general traffic engineering improvements designed to expedite overall traffic flow.

This subsection includes the following topics:

- Bus lanes; and
- Bus streets.

Bus Lanes

Bus lanes are usually used exclusively by buses; however, in some instances carpools, taxis, or turning vehicles may share the lane. Bus lanes may be located along curbs or in medians and may operate with, or counter to, automobile flow. For more information on bus lanes, refer to *Bus Lanes/Bus Rapid Transit Systems on Highways: Review of the Literature, California Partners for Advanced Transit and Highways, Berkeley (Miller, 2009)*.

Curb Bus Lanes (Normal Flow). Curb bus lanes in the normal flow direction generally convert existing lanes into bus-only lanes during peak periods. They are often implemented in conjunction with removal of curb parking so that there is little adverse effect on existing street capacity. It can be challenging to enforce the bus-only lanes during those hours and therefore may produce only marginal benefits to bus flow. In addition, this type of bus lane causes conflict between right-turning vehicles and buses.

Median Bus Lanes. Median bus lanes are in effect for the duration of the day. Wide medians are required to provide refuge for bus patrons, and passengers are required to cross active streets to reach bus stops. Additionally, left-turn traffic must be prohibited or controlled to minimize interference between transportation modes.

Bus Streets

Reserving entire streets for the exclusive use of buses represents a major commitment to transit and generally is not feasible due to adverse effects on abutting properties and businesses, including parking garages or lots, drive-in banks, etc.

Section 3 — Suburban Roadways

Overview

The term "suburban roadway" refers to high-speed roadways (speed limit or operating speeds of at least 50 mph is expected) that serve as transitional roads between low-speed urban streets and high-speed rural highways. Suburban roadways are typically 1 to 3 miles in length and have light to moderate driveway densities (approximately 10 to 30 driveways per mile). Because of their location, suburban roadways have both rural and urban characteristics. For example, these sections may maintain high speeds (a rural characteristic) while utilizing curb and gutter to facilitate drainage (an urban characteristic). Consequently, guidelines for suburban roadways typically fall between those for rural highways and urban streets.

Basic Design Features

This subsection includes information on the following basic design features for suburban roadways:

- Geometric Design Criteria for Suburban Roadways;
- Access Control;
- Medians;
- Median Openings;
- Clear Zone;
- Borders;
- Grade Separations and Interchanges;
- Right of Way Width;
- Intersections;
- Speed Change Lanes; and
- Parking.

Table 3-5 shows tabulated basic geometric design criteria for suburban roadways. The basic design criteria shown in this table reflect minimum and desired values that are applicable to new location, reconstruction or major improvement projects.

See Chapter 2 for additional guidance on choosing an appropriate design speed.

Table 3-5: Geometric Design Criteria for Suburban Roadways

Item	Functional Class	Desirable	Minimum
Design Speed (mph)	All	60	50
Horizontal Radius	All	See Table 2-4 and 2-5	
Maximum Grade (%)	All	See Table 2-9	
Stopping Sight Distance	All	See Table 2-1, Figure 2-3	
Width of Travel Lanes (ft.)	Arterial Collector Local	12 12 12	11[1] 11[2] 11[2]
Curb Parking Lane Width (ft.)	All	None	
Shoulder Width (ft.)[6,7]	All	10	4
Width of Speed Change Lanes(ft.)[3]	All	12	10
Offset to Face of Curb (ft.)	All	2	1
Median Width	All	See Medians, Urban Streets	
Border Width (ft.)	Arterial Collector Local	20 20 10	15 15 10
Right-of-Way Width (ft.)	All	Variable[4]	
Sidewalk Width (ft.)	All	6-8[5]	5
Superelevation	All	See Chapter 2, Superelevation Rate, Superelevation Transition Length, Superelevation Transition Placement, Superelevation Transition Type	
Clear Zone	All	See Table 2-12	
Vertical Clearance for New Structures (ft.)	All	See Table 2-11	
Turning Radii	All	See Chapter 7, Minimum Designs for Truck and Bus Turns	

Table 3-5: Geometric Design Criteria for Suburban Roadways

Item	Functional Class	Desirable	Minimum
Notes: 1. In highly restricted locations, 10-ft permissible. 2. In industrial areas 12-ft usual, and 11-ft minimum for restricted right-of-way conditions. In non-industrial areas, 10-ft minimum. 3. Applicable when right or left-turn lanes are provided. 4. Right-of-way width is a function of roadway elements as well as local conditions. 5. Applicable for commercial areas, school routes, or other areas with concentrated pedestrian traffic. 6. A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated. Off-system Bridges, with current ADT greater than 400 ADT, where this addition may represent an unreasonable increase in cost may be excepted from the bicycle clear space requirement. See Ch. 6 Section 1 for specific off-system bridge requirements for current ADT of 400 or less. 7. Where right turn lanes are present on uncurbed facilities, a 4-ft fully surfaced shoulder must be provided.			

Access Control

A major concern for suburban roadways is the large number of access points introduced due to commercial development. These access points create conflicts between exiting/entering traffic and through traffic. The potential for severe crashes is also increased due to the high-speed differentials. Driver expectancy can be violated because through traffic drivers traveling at high speeds do not expect to have to slow down or stop. Research has shown that reducing the number of access points and increasing the amount of access control will reduce the potential for crashes. In addition, potential crash experience can be reduced by separating conflicting traffic movements with the use of turn bays and/or turn lanes. See TxDOT's *Access Management Manual* for additional information.

Medians

Medians are desirable for suburban roadways with four or more lanes to provide storage space for left-turning vehicles. The types of medians used on suburban roadways include raised medians with mountable curb and two-way left-turn lanes.

Raised Medians. See Raised Medians, Urban Streets.

Two-Way Left-Turn Lanes. Two-way left-turn lanes (TWLTL) are applicable on suburban roadways with moderate traffic volumes and low to moderate demands for left turns. For suburban roadways, TWLTL facilities should be between 14-ft and 16-ft wide.

The desired value of 16-ft width should be used on new location projects or on reconstruction projects where widening necessitates the removal of exterior curbs. The minimum width of 14-ft is appropriate for restrictive right-of-way projects and improvement projects where attaining desirable median width would necessitate removing and replacing exterior curbing to gain only a small amount of roadway width.

A site can be considered suitable for the use of a TWLTL when a suburban roadway meets the following criteria:

- Future ADT volume of greater than 3,000 vehicles per day for an existing two-lane suburban roadway, 6,000 vehicles per day for an existing four-lane suburban roadway, or 10,000 vehicles per day for an existing six-lane suburban roadway; and
- Side roads plus driveway density of 10 or more entrances per mile.

When the above two conditions are met, the site should be considered suitable for the use of a TWLTL.

All cross sections should be evaluated for pedestrian crossing capabilities.

Median Openings

As the number of median openings along a suburban roadway increase, the interference between through traffic and turning traffic increases. To reduce the conflicts between turning traffic and through traffic, turn bays should be provided at all median openings. Recommended minimum median opening spacings are based on the length of turn bay required. For additional information regarding the design of median openings, see Section 2, Urban Streets, Medians.

Clear Zones

Table 2-12 presents the general clear zone guidelines for suburban roadways.

Borders

See Borders Urban Streets.

Grade Separations and Interchanges

See Grade Separations and Interchanges, Freeways; Grade Separations and Interchanges, Urban Streets; and Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*.

Right-of-Way Width

Right-of-way width is the area necessary to accommodate the various cross-sectional elements, including widths of travel and turning lanes, bicycle lanes, shoulders, median, sidewalks, sidewalk offsets, slopes, and borders. The width of right-of-way for suburban roadways is influenced by traffic volume requirements, lane use, cost, extent of ultimate expansion, and land availability.

Intersections

Due to high operating speeds (50 mph or greater) on suburban roadways, curve radii for turning movements should equal that of rural highway intersections. Space restrictions due to right-of-way limitations in suburban areas may necessitate reduction in the values given for rural highways. For additional information regarding intersection design, see Intersections Urban Streets.

Speed Change Lanes

Speed Change Lanes are defined as acceleration or deceleration lanes for left or right turns, exit or entrance acceleration or deceleration lanes, or a climbing lane. A design waiver is required for speed change lanes with lengths and widths that do not meet minimum criteria.

Speed change lanes may be provided as space for deceleration/acceleration to/from intersecting side streets with significant volumes and high operating speeds. For information regarding the design of left-turn (median) speed change lanes and right speed change lanes, see Section 2, Urban Streets, Speed Change Lanes. (See Table 3-3 for lengths of single left-turn lanes; Table 3-4 for lengths of dual left-turn lanes, Figure 3-5 for length of right-turn lanes.)

Parking

Parking adjacent to the curb on suburban roadways should not be allowed.

Section 4 — Two-Lane Rural Highways

Overview

The term "two-lane rural highway" as used in this chapter, refers to roadways in un-developed areas that have one-lane of traffic in each direction. Access to these facilities is controlled through driveway locations and intersecting roadways. Two-lane rural highways usually do not have any type of median barrier.

Basic Design Features

This subsection includes information on the following basic design features for two-lane rural highways:

- Geometric Design Criteria for Two-Lane Rural Highways;
- Access Control;
- Transitions to Four-Lane Divided Highways;
- Passing Sight Distances;
- Speed Change Lanes; and
- Intersections.

Additional information on structure widths may be obtained in TxDOT's *Bridge Design - LRFD* and the *Bridge Project Development Manual*.

Table 3-6 shows references for the geometric design criteria for two-lane rural highways.

Table 3-6. Geometric Design Criteria for Rural Two-Lane Highways

Geometric Design Element	Reference or Design Value
Design Speed	Table 3-7
Minimum Horizontal Radius	Table 2-4, Table 2-5
Maximum Grade (%)	Table 2-9
Stopping Sight Distance	Table 2-1, Figure 2-3
Width of Travel Lanes	Table 3-8
Width of Shoulders	Table 3-8
Vertical Clearance for New Structures	Table 2-11
Clear Zone	Table 2-12
Passing Sight Distance	Table 3-9

Table 3-6. Geometric Design Criteria for Rural Two-Lane Highways

Geometric Design Element	Reference or Design Value
Superelevation	See Chapter 2, Superelevation Rate, Superelevation Transition Length, Superelevation Transition Placement, Superelevation Transition Type
Turning Radii	See Chapter 7, Minimum Designs for Truck and Bus Turns

Table 3-7 shows minimum design speed for rural two-lane highways. See Chapter 2 for definition of level and rolling terrain as well as guidance on choosing an appropriate design speed.

Table 3-7: Minimum Design Speed for Rural Two-lane Highways

Functional Class	Terrain	Minimum Design Speed (mph) for future ADT of:		
		< 400	400-2000	> 2000
Arterial	Level	70		
	Rolling	60		
Collector	Level	50[1]	50	60
	Rolling	40[2,4]	40[4]	50
Local[3]	Level	40[2,4]	50	50
	Rolling	30[4]	40[4]	40[4]

Notes:
1. A 40 mph minimum design speed may be used where roadside environment or unusual design considerations dictate (e.g., significant horizontal curvature due to mountainous or hilly terrain).
2. A 30 mph minimum design speed may be used where roadside environment or unusual design considerations dictate (e.g., significant horizontal curvature due to mountainous or hilly terrain).
3. Applicable only to off-system routes that are not functionally classified at a higher classification.
4. When determining applicable radii and superelevation, Table 2-3 and Table 2-4 (for high-speed and non-urban conditions) should be used even though listed speeds are considered low-speed.

Table 3-8: Width of Travel Lanes and Shoulders on Rural Two-lane Highways

Functional Class	Design Speed (mph)	Minimum Width [1,2,9,10](ft) for future ADT of:			
		< 400	400-1500	1500-2000	> 2000
Arterial	LANES (ft)[12]				
	All	12			
	SHOULDERS (ft)[3]				
	All	4[7]	6	8	10[8]

Table 3-8: Width of Travel Lanes and Shoulders on Rural Two-lane Highways

Functional Class	Design Speed (mph)	Minimum Width [1,2, 9, 10] (ft) for future ADT of:			
		< 400	400-1500	1500-2000	> 2000
Collector	LANES (ft)[12]				
	30	10	11	11	12
	35	10	11	11	12
	40	10	11	11	12
	45	10	11	11	12
	50	10	11	12	12
	55	10	11	12	12
	60	11	11	12	12
	65	11	11	12	12
	70	11	11	12	12
	75	11	12	12	12
	80	11	12	12	12
	SHOULDERS (ft)[5]				
	30-45	2[4, 7]	4[7]	8	10[8]
	50-80	2[4, 7]	6	8	10[8]
Local[6]	LANES (ft)[12]				
	30-50	10	11	11	12
	SHOULDERS (ft)[5]				
	All	2[7]	4[7, 11]	4[7, 11]	8[11]

Notes:

1. Minimum surfacing width is 24-ft for all on-system state highway routes.
2. On high riprapped fills through reservoirs, a minimum of two 12-ft lanes with 8-ft shoulders should be provided for roadway sections. For arterials with 2,000 or more ADT in reservoir areas, two 12-ft lanes with 10-ft shoulders should be used.
3. On arterials, shoulders fully surfaced.
4. On collectors, use minimum 4-ft shoulder width at locations adjacent to roadside barrier.
5. Shoulders must be fully surfaced on collectors with 1,500 or more ADT. Shoulder surfacing not required but is desirable even if only for partial width, on collectors with less than 1,500 ADT and all local roads.
6. Applicable only to off-system routes that are not functionally classified at a higher classification.
7. A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated. Off-system Bridges, with current ADT greater than 400 ADT, where this addition may represent an unreasonable increase in cost may be excepted from the bicycle clear space requirement. See Ch. 6 Section 1 for specific off-system bridge requirements for current ADT of 400 or less.
8. In highly constrained conditions 8-ft may be used.
9. Minimum width of new or widened structures should accommodate the approach roadway including shoulders.
10. A minimum surfacing width of 36-ft should be considered on horizontal curves with a radius less than 1910-ft (i.e. greater than 3 degrees of curvature). See FHWA *Handbook for Designing Roadways for the Aging Population* for further recommendations on minimum pavement width in horizontal curves when considering older drivers.
11. Mailbox Turnouts should be considered on local roads with 400 or more ADT. Refer to TxDOT standard drawings for details regarding the design and construction of mailbox turnouts.
12. See Ch. 3 Section 3 for TWLTL criteria.

Access Control

The installation of access driveways along two-lane rural highways must be in accordance with the *Access Management Manual*.

Frontage roads or parallel service roads to serve small rural business communities or other developments should not be permitted along two-lane rural highways. To a driver unfamiliar with the local area, a frontage road takes on the appearance of a separate roadway of a multilane divided facility, thus resulting in the assumption that the two-way, two-lane highway is a one-way roadway. Where individual driveways are located within deep cut or high fill areas, driveways may be routed parallel to the highway for short distances to provide for a safe, economical junction with the highway.

Transitions to Four-Lane Divided Highways

Typical transitions from two-lane to four-lane divided highways are discussed in Transitions to Four-Lane Divided Highways, Multi-Lane Rural Highways, and illustrated in Figure 3-16.

Passing Sight Distances

Passing sight distance is the distance required by a driver to make a passing maneuver without cutting off the passed vehicle and before meeting an opposing vehicle. Therefore, passing sight distance is applicable to two-lane highways only (including two-way frontage roads).

Recommended passing sight distances are based on the following assumptions:

- The speeds of the passing and opposing vehicles are equal and represent the design speed of the highway;
- The passed vehicle travels at a uniform speed and the speed difference between the passing and passed vehicles is 12 mph;
- The passing vehicle has sufficient acceleration capability to reach the specified speed difference relative to the passed vehicle by the time it reaches the critical position, which generally occurs about 40 percent of the way through the passing maneuver;
- The lengths of the passing and passed vehicles are 19-ft;
- The passing driver's perception-reaction time in deciding to abort passing a vehicle is 1-s;
- If the passing maneuver is aborted, the passing vehicles will use a deceleration rate of 11.2 ft/s2, the same deceleration rate used in stopping sight distance design criteria;
- For a completed or aborted pass, the space headway between passing and passed vehicles is 1-s; and
- The minimum clearance between passing and opposing vehicles at the point at which the passing vehicle return to its normal lane is 1-s.

In the design of two-lane highways, minimum or greater passing sight distance should be provided wherever practical, since less than minimum distances reduces the safety and level of service of the roadway. For rolling terrain, provision of climbing lanes may be a more economical alternative than achieving a vertical alignment with adequate passing sight distance.

The minimum passing sight distance for a two-lane road is about twice the minimum stopping sight distance at the same design speed. To meet those greater sight distances, clear sight areas on the inside of curves should be provided. For cut sections, designing for passing sight distance should be limited to tangents and very flat curves. Even in level terrain, providing passing sight distance would need a clear area inside each curve that would, in some instances, extend beyond the normal right-of-way line.

Minimum passing sight distance values for design of two-lane highways are shown in Table 3-9. These distances are for design purposes only and should not be confused with other distances used as warrants for striping no-passing zones as shown in the *Texas Manual on Uniform Traffic Control Devices* (TMUTCD). For the design of typical two-lane rural highways, except for level terrain, provision of near continuous passing sight distance is impractical. However, the designer should attempt to increase the length and frequency of passing sections where economically feasible.

Table 3-9: Passing Sight Distance

K-Values for Determining Length of Crest Vertical Curve for Various Passing Sight Distances (PSD)		
Design Speed (mph)	**Minimum PSD for Design (ft)**	**Minimum K-Value[1]**
20	400	57
25	450	72
30	500	89
35	550	108
40	600	129
45	700	175
50	800	229
55	900	289
60	1000	357
65	1100	432
70	1200	514
75	1300	604
80	1400	700

Note:

1. Based on eye height = 3.5 ft and object height = 3.5 ft; $K = PSD^2 / 2800$.

Speed Change Lanes

Speed Change Lanes are defined as acceleration or deceleration lanes for left or right turns, exit or entrance acceleration or deceleration lanes, or climbing lanes. A design waiver is required for speed change lanes that do not meet minimum length or width criteria.

Climbing Lanes. It is desirable to provide an extra lane on the upgrade side of a two-lane highway as a climbing lane where the grade, traffic volume, and heavy vehicle volume combine to degrade traffic operations.

A climbing lane should be considered when one of the following three conditions exist:

- 10 mph or greater speed reduction is expected for a typical heavy vehicle;
- Level-of-service E or F exists on the upgrade; or
- A reduction of two or more levels of service is experienced when moving from the approach segment to the upgrade.

  For low-volume roadways there is minimal delay and a climbing lane may not be justified. For this reason, a climbing lane should only be considered on roadways with the following traffic conditions:

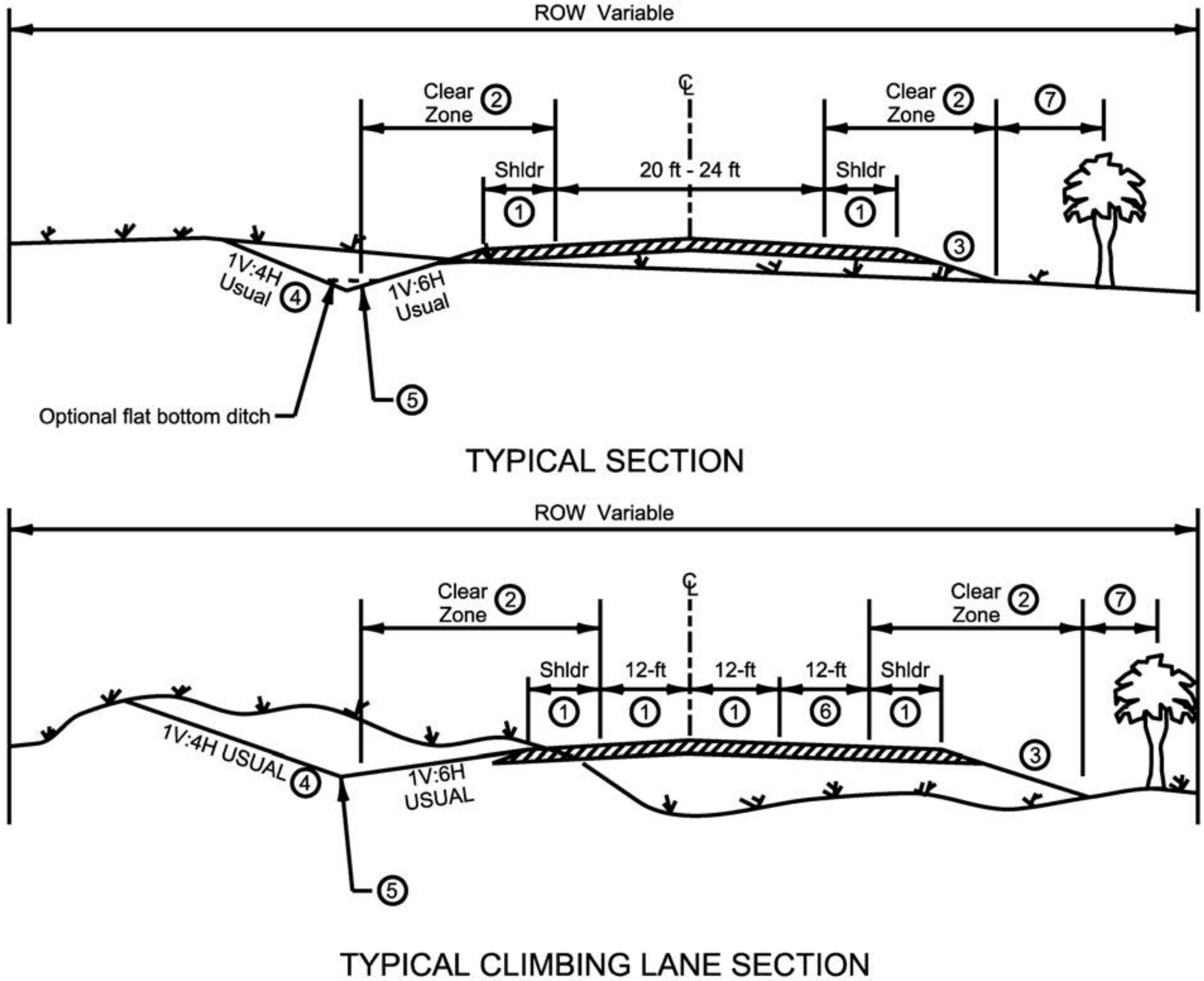
- Upgrade traffic flow rate is greater than 200 vehicles per hour or
- Upgrade truck flow rate is greater than 20 vehicles per hour.

The upgrade flow rate is estimated by multiplying the anticipated or existing design hour volume by the directional distribution factor for the upgrade direction and dividing the result by the peak hour factor (see Traffic Characteristics, Chapter 2 and the *Highway Capacity Manual* for definitions of these terms). To calculate the upgrade truck flow rate, multiply the upgrade flow rate by the percentage of trucks in the upgrade direction.

The beginning of a climbing lane should be introduced near the foot of the grade. The climbing lane should be preceded by a tapered section with a desirable taper ratio of 25:1 that should be at least 300-ft long.

Attention should also be given to the location of the terminal point of the climbing lane. Ideally, the climbing lane should be extended to a point beyond the crest where a typical truck could reach a speed that is within 10 mph of the speed of other vehicles. In addition, climbing lanes should not end just prior to an obstruction such as a restrictive width bridge. The climbing lane should be followed by a tapered section with a ratio of 50:1.

For projects in new locations, or where an existing highway will be regraded, consider improving the grade line in lieu of providing a climbing lane. Refer to Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets* for more information regarding the design of climbing lanes. Figure 3-7 shows a cross section for climbing lanes on rural highways.



Notes:

1. For widths of travel lanes and shoulders, see Table 3-8.
2. See Table 2-12
3. See Chapter 2, Section 6 for side slope rates
4. Slope may be exceeded in rock cuts, for restricted right of way or deep cut conditions, or where ditch is not within the horizontal clearance requirements.
5. See discussion of preferred ditch sections in Chapter 2, Section 6.
6. See Table 3-1 (Width of Speed Change Lanes) for climbing lane widths.
7. Consideration should be given to removing fixed objects 10ft beyond the toe of slope.

Figure 3-7. Cross Sections for Arterial and Collector Two-Lane Rural Highways.

Left-Turn Deceleration Lanes. The additional expense of adding left-turn lanes on two-lane highways at intersecting cross roads is often not justified due to low volumes. For certain moderate or high-volume two-lane highways with heavy left-turn movements, however, left-turn lanes may be justified in view of reduced road user crash costs. Table 3-10 provides recommendations for when left-turn lanes should be considered for a typical two-lane highway intersection. Lengths of left-turn deceleration lanes are provided in Table 3-12.

In instances on three-leg intersections where a left turn lane is not warranted due to low major roadway volume, but separation of through and turning traffic is still desired due to moderate to high left turn volume, a bypass lane can be installed (see Figure 3-8). For width of left-turn deceleration lanes see Table 3-1.

Table 3-10. Guide for Left-Turn Lane Warrants on Two-Lane Highways in Rural Areas[1]

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection Major-Road Peak-Hour Volume (veh/hr/ln) for a Bypass Lane	Three-Leg Intersection Major-Road Peak-Hour Volume (veh/hr/ln) for a Left-Turn Lane	Four-Leg Intersection Major-Road Peak-Hour Volume (veh/hr/ln) for a Left-Turn Lane
5	50	200	150
10	50	100	50
15	< 50	100	50
20	< 50	50	< 50
25	< 50	50	< 50
30	< 50	50	< 50
35	< 50	50	< 50
40	< 50	50	< 50
45	< 50	50	< 50
50 or More	< 50	50	< 50

Note:

1. These guidelines apply where the major road is uncontrolled and the minor-road approaches are stop- or yield-controlled. Both the left-turn peak-hour volume and the major-road volume warrants should be met as shown in Figure 3-8.

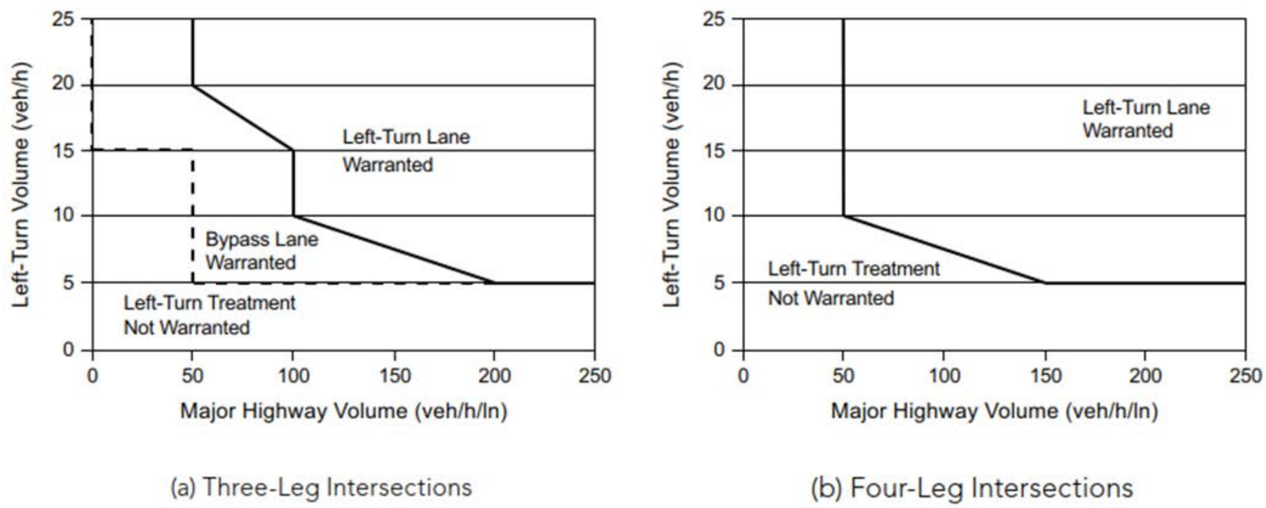


Figure 3-8. Suggested Left-Turn Warrants Based on Results from Benefit-Cost Evaluations for Intersections on Two-Lane Highways in Rural Areas.
Source: AASHTO's A Policy on Geometric Design of Highways and Streets

Example:

Three Leg Intersection

Left Turn Volume = 17 veh/hr

Major Road Volume = 150 veh/hr; 2 ln

Choose the next highest turn lane volume of 20 veh/h. The major road is 75 veh/h/ln, which is greater than 50, therefore a left turn lane is warranted.

Where used, left-turn lanes should be delineated with striping and pavement markers or jiggle bars. Passing should be restricted in advance of the intersection, and horizontal alignment shifts of the approaching travel lanes should be gradual. Figure 3-9 shows typical geometry for a rural two-lane highway with left-turn bays at an intersecting crossroad.

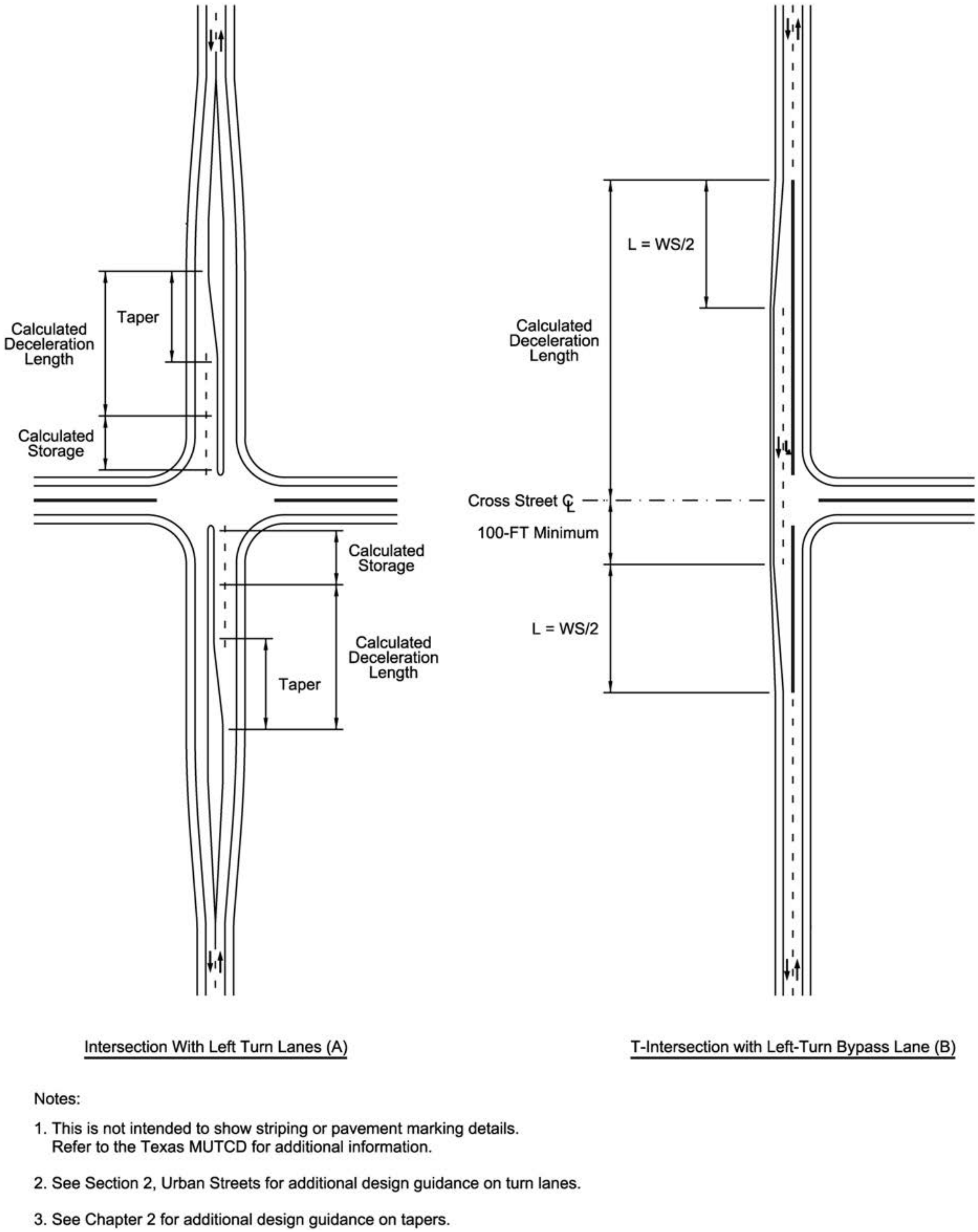


Figure 3-9. Typical Two-Lane Highway Intersection with Left-Turn Lanes.

Right-Turn Deceleration Lanes. Shoulders 10-ft wide alongside the traffic lanes generally provide sufficient area for acceleration or deceleration of right-turning vehicles. Where the right turn deceleration or acceleration lane is being constructed adjacent to the through lanes, the minimum lane width is 10-ft with a 2-ft surfaced shoulder. Speed change lanes should be symmetrical along both sides of the highway to provide drivers with a balanced section.

A deceleration-acceleration lane on one side of a two-lane highway, such as at a "tee" intersection, results in the appearance of a three-lane highway and may result in driver confusion. Therefore, right-turn speed change lanes are generally inappropriate for "tee" intersection design except where a four-lane section is provided. An example of this configuration is two through lanes, one median left turn lane and one right acceleration/deceleration lane.

Figure 3-5 shows an example of right-turn deceleration lanes.

The length of a right-turn deceleration lane is the same as that for a left-turn lane (see Table 3-12). On some low-volume rural highways, it may be acceptable to provide right turn lanes shorter than the lengths given in Table 3-12.

Right-Turn Acceleration Lanes. Right-turn acceleration lanes may be appropriate on some two-lane rural highways such as high-volume highways where significant truck percentages are encountered. See Table 3-13 for acceleration distances and taper lengths.

Grade Separations and Interchanges

See Grade Separations and Interchanges, Freeways and Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*.

Intersections

The provision of adequate sight distance is of utmost importance in the design of intersections along two-lane rural highways. At intersections, consideration should be given to avoid steep profile grades and limited horizontal or vertical sight distance. An intersection should not be situated just beyond a short crest vertical curve or a sharp horizontal curve. Where necessary, backslopes should be flattened and horizontal and vertical curves lengthened to provide additional sight distance. For more information on intersection sight distance, see Intersection Sight Distance in Chapter 2.

The roadways should intersect at approximately right angles and should not intersect less than 75 degrees. Where crossroad skew is less than 75 degrees to the highway, the crossroad should be realigned to provide for a near perpendicular crossing. As a general rule, the higher the functional classification, the closer the crossroad intersection should be to 90 degrees.

Chapter 7 provides additional information regarding the accommodation of various types of truck class vehicles in intersection design in the section on Minimum Designs for Truck and Bus Turns. Further information on intersection design may also be found in AASHTO's *A Policy on Geometric Design of Highways and Streets*.

Transverse or In-Line Rumble Strips. See Chapter 7 Section 5 Rumble Strips for additional traffic calming measures at intersections.

Section 5 — Multi-Lane Rural Highways

Overview

The term “multi-lane rural highway” as used in this chapter, refers to roadways in un-developed areas that have more two or more lanes of traffic in each direction. Access to these facilities is controlled through driveway locations and intersecting roadways. Multi-lane rural highways can be undivided or divided with a depressed median or a surface mounted median.

Basic Design Features

This subsection includes guidelines on geometric features for multilane rural highways. The guidelines are outlined in Table 3-11, Figure 3-10 and Figure 3-11. These guidelines apply for all functional classes of roadways.

Table 3-11 shows minimum design speed for rural multilane highways. See Chapter 2 for definition of level and rolling terrain as well as guidance on choosing an appropriate design speed.

Table 3-11: Design Criteria for Multilane Rural Highways (Non-controlled Access) (All Functional Classes)

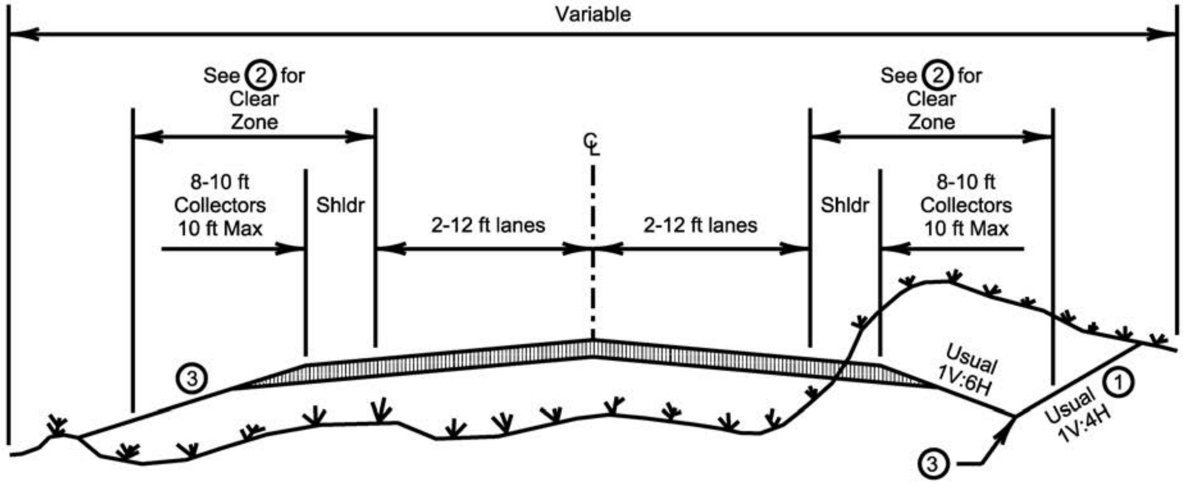
Type of Facility		Six-Lane Divided		Four-Lane Divided		Four-Lane Undivided	
Design Speed (Arterials) (mph)		Min.		Min.		Min.	
Level		70[1]		70[1]		70[1]	
Rolling		60[2]		60[2]		60[2]	
Design Speed (Collector) (mph)		Min.		Min.		Min.	
Level		60		60		60	
Rolling		50		50		50	
Lane Width (ft)[6]		12		12		12	
-		Des.	Min.	Des.	Min.	Des.	Min
Median Width (ft)	Surfaced	16	4	16	4	Not Applicable	
-	Depressed	76	48	76	48	-	
Shoulder Outside (ft)[4,5]		10	8	10	8	10	8[3]
Shoulder Inside (ft) for Depressed Medians[5]		10	4	4	4	Not applicable	
Vertical Clearance, New Structures		Table 2-11					
Minimum Horizontal Radius		Table 2-4 and Table 2-5					

Table 3-11: Design Criteria for Multilane Rural Highways (Non-controlled Access) (All Functional Classes)

Type of Facility	Six-Lane Divided	Four-Lane Divided	Four-Lane Undivided
Stopping Sight Distance	Table 2-1		
Maximum Grade (%)	Table 2-9		
Clear Zone	Table 2-12		
Superelevation	See Chapter 2, Superelevation Rate, Superelevation Transition Length, Superelevation Transition Placement, Superelevation Transition Type		
Turning Radii	See Chapter 7, Minimum Designs for Truck and Bus Turns		

Notes:

1. 60 mph acceptable when conditions warrant and are documented through a design exception.
2. 50 mph acceptable when conditions warrant and are documented through a design exception.
3. On four-lane undivided collector highways, outside surfaced shoulder width may be decreased to 4-ft where flat (1V:10H), sodded front slopes are provided for a minimum distance of 4-ft from the shoulder edge. Arterials must have a fully surfaced shoulder.
4. A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated. Off-system Bridges, with current ADT greater than 400 ADT, where this addition may represent an unreasonable increase in cost may be excepted from the bicycle clear space requirement. See Ch. 6 Section 1 for specific off-system bridge requirements for current ADT of 400 or less.
5. Where left or right turn lanes are present on uncurbed facilities, a 4-ft fully surfaced shoulder must be provided.
6. See Ch. 3 Section 3 for TWLTL criteria.

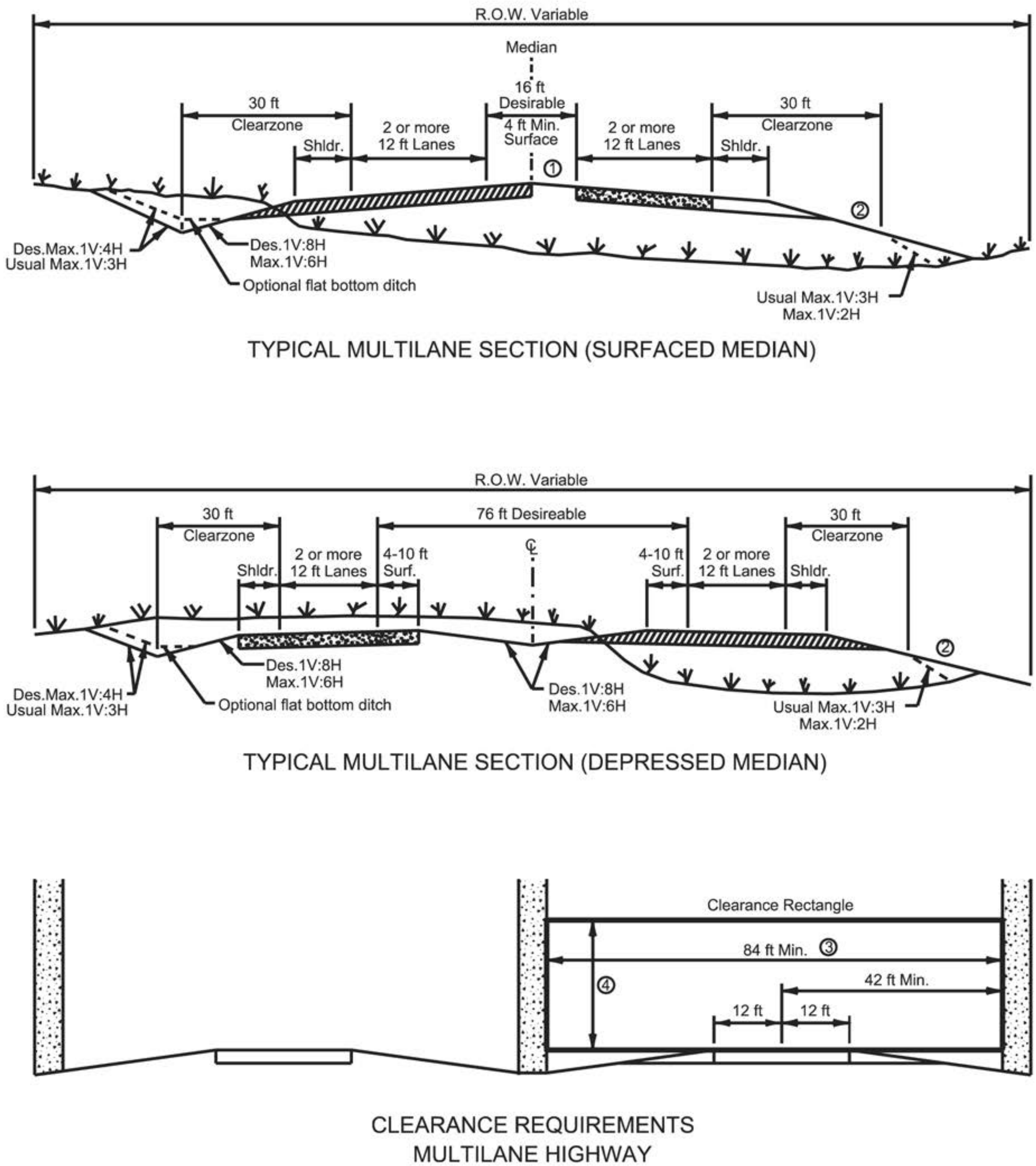


TYPICAL MULTILANE SECTION (Undivided Highway)

Notes:

1. Slope may be exceeded in rock cuts, for restricted right of way or deep cut conditions, or where ditch is not within horizontal clearances.
2. See Table 2-12.
3. See Chapter 2, Section 6 for side slope rates and discussion of preferred ditch sections.

Figure 3-10. Cross Sections for Arterial and Collector Multi-Lane Undivided Rural Highways.



① Medians full pavement structure.

② For side slope rates, see Chapter 2, Slopes and Ditches.

③ Where future lanes are anticipated, add 12 ft to width of clearance rectangle for each added lane.

④ Refer to Chapter 2, Vertical Clearance.

Figure 3-11. Cross Sections for Multi-Lane Rural Highways.

Level of Service

Rural arterials and their auxiliary facilities should be desirably designed for Level of Service B in the design year as defined in the *Highway Capacity Manual*.

Generally, undivided four-lane roadways have been associated with higher crash rates than divided roadways. This higher crash rate has frequently been attributed to the lack of protection for left-turning vehicles. Therefore, if an undivided facility is selected for a location, the impact of left-turning vehicles should be examined.

For more information regarding level of service as it relates to facility design, see Service Flow Rate in the sub section titled Traffic Volume of Chapter 2, Section 3.

Basic Design Features

This subsection includes information on the following basic design features for multi-lane rural highways:

- Access Control;
- Medians;
- Turn Lanes;
- Travel Lanes and Shoulders;
- Intersections;
- Transitions to Four-Lane Divided Highways; and
- Grade Separations and Interchanges.

Access Control

The installation of all access driveways along multilane facilities from adjacent property connecting to the mainlanes should be in accordance with the TxDOT Access Management *Manual*.

For multilane highways constructed in developed or developing areas, it may be desirable to control access to the mainlanes through right-of-way acquisition or by design (i.e., provision of frontage roads). Designed access control may be provided solely in the interchange areas or continuously throughout a section of highway, depending on traffic volumes, the degree of roadside development, availability of right-of-way, and economic conditions.

All frontage road development must be in accordance with the rules contained in 43 Texas Administrative Code (TAC) §15.54. The *Project Development Process Manual* can also be referenced for additional information.

Medians

The width of the median in a multi-lane rural highway is the distance between the inside edges of the opposing travel lanes. If practical, wide medians (approximately 76-ft) should be used to provide sufficient storage space for tractor-trailer combination vehicles at median openings, reduce headlight glare, provide a pleasing appearance, reduce the chances of head-on collisions and provide a sheltered storage area for crossing vehicles, including tractor-trailer combinations. Wide medians should generally be used whenever feasible but median widths greater than 60-ft have been found to be undesirable for intersections that are signalized or may be signalized in the design life of the project.

In areas that are likely to become suburban or urban in nature, medians wider than 60-ft should be avoided at intersections except where necessary to accommodate turning and crossing maneuvers by larger vehicles. Wide medians may be a disadvantage when signalization is required at future intersections. The increased time for vehicles to cross the median can lead to inefficient signal operation.

Four-Lane Undivided Highways. Conversion of a two-lane highway to a four-lane highway facility should include a median when possible. If an existing two-lane highway has rolling terrain or restricted right-of-way conditions, conversion to a four-lane undivided highway may be considered to improve passing opportunities and traffic operations. Table 3-11 and Figure 3-10 include the general geometric features for four-lane undivided highways. In cases where a median is being proposed and the existing roadbed will remain in place, Non-Freeway Rehabilitation (3R) alignment criteria may be applied to the existing roadbed as described in Chapter 4. However, 4R criteria must be applied to the new roadbed.

Surfaced Medians. Surfaced median designs are most appropriate in areas with roadside development. Surfaced medians of 4-ft to 16-ft are classified as narrow medians and are used in restricted conditions. Medians 4-ft wide provide little separation of opposing traffic and a minimal refuge area for pedestrians. Surfaced medians of 14-ft to 16-ft offer space for use by exiting traffic turning left, but do not offer protection for crossing vehicles.

Median Openings. Closely spaced median openings on divided highways can cause interference between high-speed through-traffic and turning vehicles. The frequency of median openings varies with topographic restrictions and local requirements As a general rule, the minimum spacing should be one-quarter mile or greater in rural areas. It is typical to provide median openings at all public roads and at major traffic generators such as industrial sites or shopping centers. Additional openings should be provided to maintain a maximum one-half mile spacing.

As shown in Figure 3-12, left-turn lanes should be provided at all median openings and right-turn deceleration and acceleration lanes should be considered at intersections with highways or other major public roads with significant turning movements. See the Access Management Manual, “Auxiliary Lanes” section and related table for additional considerations and warranting thresholds for right-turn deceleration and acceleration lanes as well as left-turn deceleration lanes at median openings.

For divided highways with independent main lane alignment, particular care should be exercised at median openings to provide a satisfactory profile along the crossover with flat approaches to the main lanes.

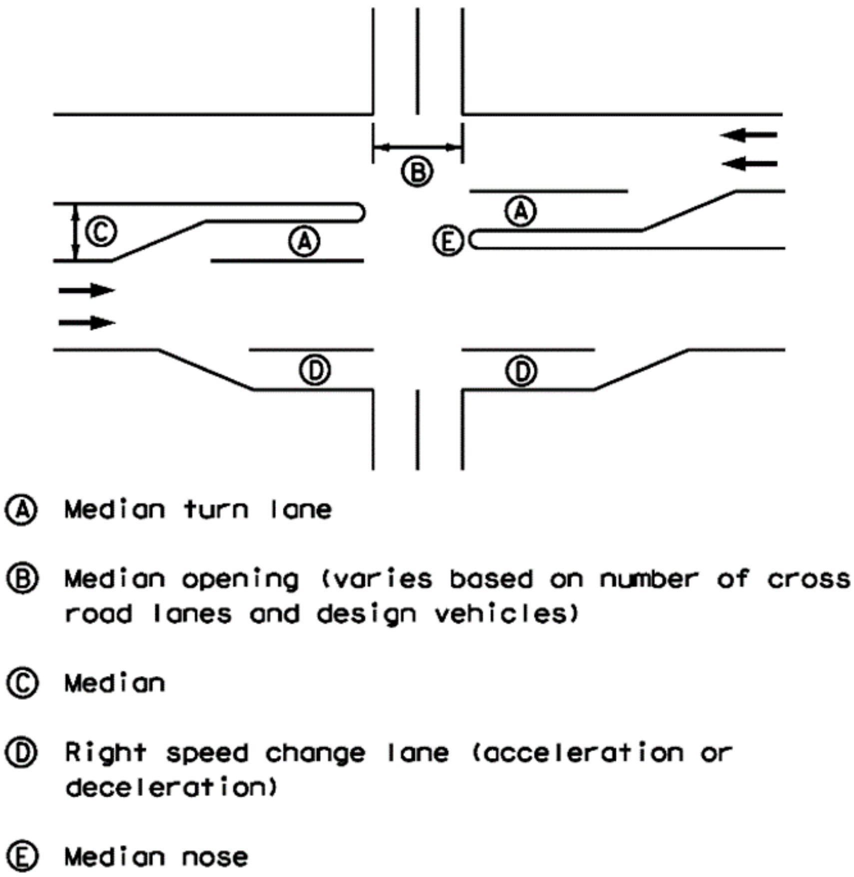


Figure 3-12. Multi-Lane Rural Highway Intersection.

Median openings should be at least 40-ft wide or the width of the crossroad pavement width plus 8-ft. Design vehicles are often used as the basis for minimum design of median openings, particularly for multilane cross roads and skewed intersections. See Chapter 7, Section 7, Minimum Designs for Truck and Bus Turns for additional information.

Turn Lanes

Turn lanes, or speed change lanes, should generally be provided wherever vehicles must slow to leave a facility or accelerate to merge onto a facility.

Median Turn Lane (Left-Turn Lane). Median turn lanes with 4-ft adjacent shoulders provide deceleration and storage area for vehicles making left turns to leave a divided highway. Storage, taper, and deceleration lengths for design are illustrated in Figure 3-13 and summarized in Table 3-12. Taper lengths shorter than those in Table 3-12 may be acceptable on some low volume rural highways. Also, adjustments for grade are given in Table 3-14

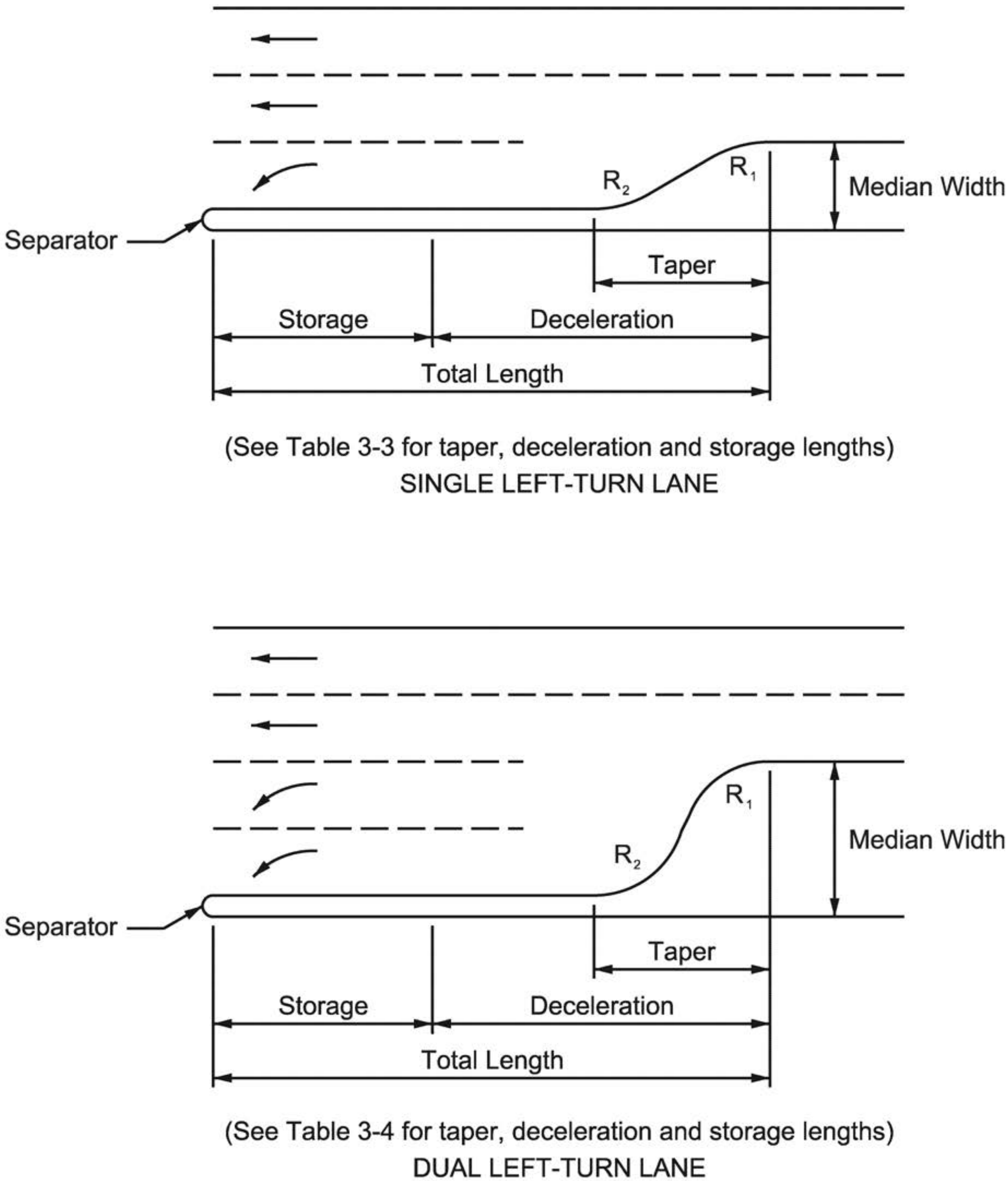


Figure 3-13. Left Turn Lanes on Multilane Rural Highways.

Table 3-12: Lengths of Median Turn Lanes Multilane Rural Highways

Mainlane Design Speed (mph)	Taper Length (ft)[1]	Deceleration Length (ft)[2]
30	50	150
35	50	205
40	50	265
45	100	340
50	100	415
55	100	505
60	150	600
65	150	700
70	150	815
75	150	935
80	150	1,060

Notes:
1. For low volume median openings, such as those serving private drives or U-turns, a taper length of 100-ft may be used regardless of mainlane design speed.
2. Based on 6.5 ft/s^2 deceleration to stopped condition throughout the entire length. Larger deceleration rates may be used when deceleration lengths based on 6.5 ft/s^2 are impractical.

Storage Length Calculations. For storage length calculations on multi-lane rural highways, the storage length calculations in Urban Streets apply.

Right Turn Lane. Right turn lanes (12-ft lane with 4-ft adjacent shoulders) provide deceleration or acceleration areas for right-turning vehicles. The deceleration length and taper lengths for right turn lanes are the same as for Median Turn lanes (see Table 3-12). Adjustment factors for grade effects are shown in Table 3-14.

Acceleration Lanes. Acceleration lanes for right-turning or left-turning vehicles may be desirable for vehicles entering on multi-lane rural highways. Examples of both tapered and parallel accelerations lanes are shown in Figure 3-14. Recommended acceleration lengths are shown in Table 3-13. Adjustments for grade are given in Table 3-14.

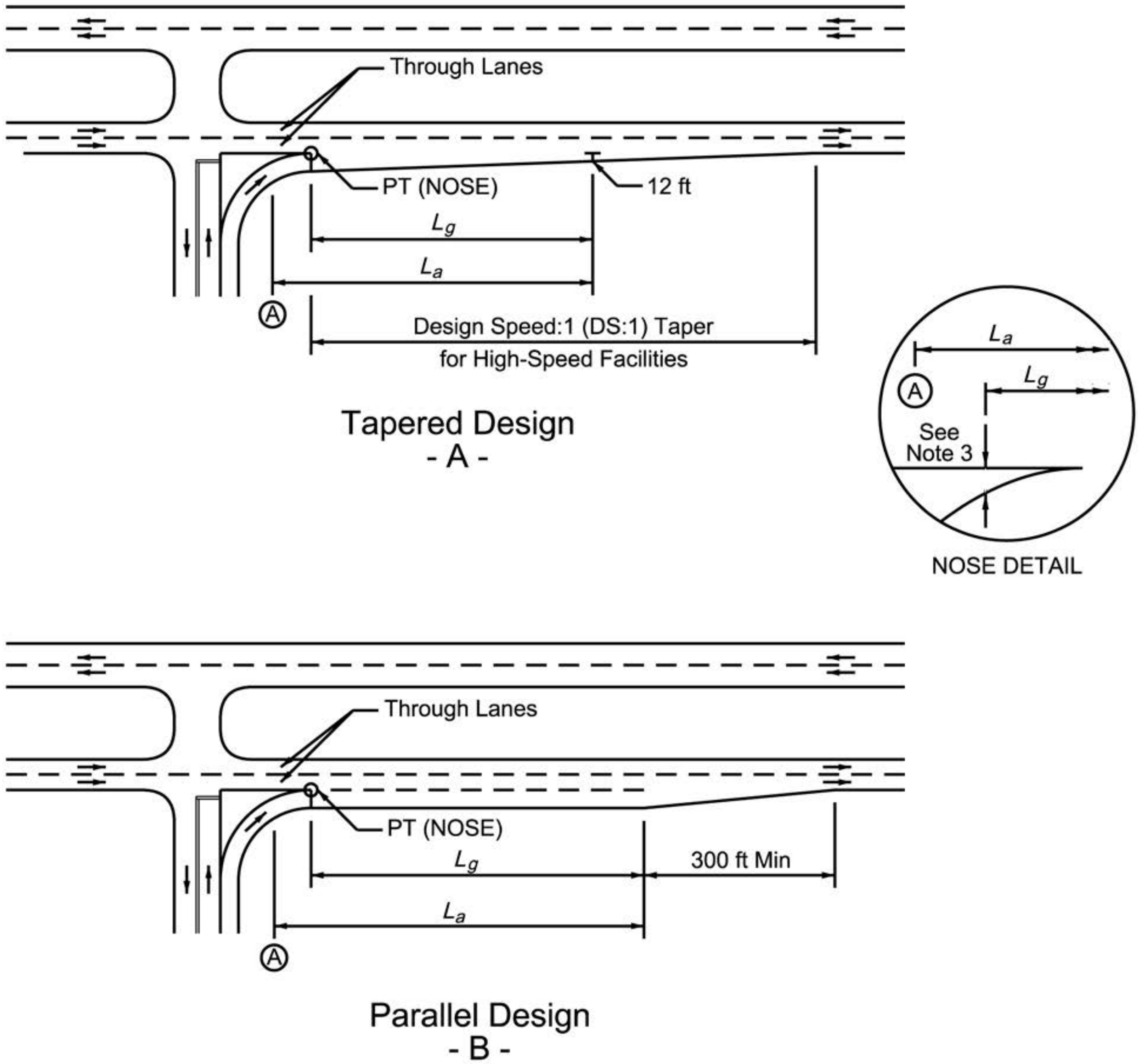


Figure 3-14. Examples of Tapered and Parallel Acceleration Lanes.

Notes:

1. L_a is the recommended acceleration length as shown in Table 3-13 or as adjusted by Table 3-14.
2. Point A is the feature that controls speed on the acceleration lane. La should not start back on the curvature of the ramp unless the radius equals 1,000-ft or more.
3. L_g is the recommended gap acceptance length. Lg should be a minimum of 300 to 500-ft depending on nose width. (Nose width 2'-10')
4. The value of L_a or L_g, whichever produces the greater distance downstream from where the nose equals 2-ft, is suggested for use in the design of the acceleration lane distance.

Table 3-13: Minimum Acceleration Lane Lengths for Entrance Terminals with Flat Grades of Less Than 3%

	Design Speed of Controlling Feature on Ramp (mph)								
Highway Design Speed (mph)	**Stop Condition**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
30	180	140	-	-	-	-	-	-	-
35	280	220	160	-	-	-	-	-	-
40	360	300	270	210	120	-	-	-	-
45	560	490	440	380	280	160	-	-	-
50	720	660	610	550	450	350	130	-	-
55	960	900	810	780	670	550	320	150	-
60	1,200	1,140	1,100	1,020	910	800	550	420	180
65	1,410	1,350	1,310	1,220	1,120	1,000	770	600	370
70	1,620	1,560	1,520	1,420	1,350	1,230	1,000	820	580
75	1,790	1,730	1,630	1,580	1,510	1,420	1,160	1,040	780
80	2,000	1,900	1,800	1,750	1,680	1,600	1,340	1,240	980

Table 3-14: Speed Change Lane Adjustment Factors as a Function of a Grade

(US Customary)								
Deceleration Lanes								
Design Speed of Roadway (mph)	**Ratio of Length on Grade to Length on Level[1]**							
	3 to 4 % Upgrade		**3 to 4 % Downgrade**		**5 to 6% Upgrade**		**5 to 6% Downgrade**	
All	0.9		1.2		0.8		1.35	
Acceleration Lanes								
Design Speed of Roadway (mph)	**Ratio of Length on Grade to Length on Level[1] for Design Speed (mph) of Turning Roadway Curve**							
	20	**25**	**30**	**35**	**40**	**45**	**50**	**All Speeds**
	3 to 4 % Upgrade							**3 to 4% Downgrade**
40	1.3	1.3	1.3	1.3	----	----	----	0.7
45	1.3	1.3	1.35	1.35	----	----	----	0.675
50	1.3	1.35	1.4	1.4	1.4	----	----	0.65
55	1.35	1.4	1.45	1.45	1.45	1.45	----	0.625
60	1.4	1.45	1.5	1.5	1.5	1.55	1.6	0.6

Table 3-14: Speed Change Lane Adjustment Factors as a Function of a Grade

(US Customary)								
65	1.45	1.5	1.55	1.55	1.6	1.65	1.7	0.6
70	1.5	1.55	1.6	1.65	1.7	1.75	1.8	0.6
75	1.6	1.65	1.7	1.75	1.8	1.9	2.0	0.6
80	1.7	1.75	1.8	1.9	2.0	2.05	2.1	0.6
	5 to 6% Upgrade							**5 to 6% Downgrade**
40	1.5	1.5	1.5	1.6	----	----	----	0.6
45	1.5	1.55	1.6	1.6	----	----	----	0.575
50	1.5	1.6	1.7	1.8	1.9	2.0	----	0.55
55	1.6	1.7	1.8	1.9	2.05	2.1	----	0.525
60	1.7	1.8	1.9	2.05	2.2	2.4	2.5	0.5
65	1.85	1.95	2.05	2.2	2.4	2.6	2.75	0.5
70	2.0	2.1	2.2	2.4	2.6	2.8	3.0	0.5
75	2.15	2.25	2.35	2.58	2.8	3.03	3.25	0.5
80	2.3	2.4	2.5	2.75	3.0	3.25	3.5	0.5

Note:

1. Ratio in this table multiplied by length of deceleration or acceleration distances in Table 3-3 and Table 3-13, gives length of deceleration/acceleration distance on grade.

Travel Lanes and Shoulders

Travel Lanes. Travel lanes should be provided with widths as shown in Table 3-11. *The Highway Capacity Manual* should be consulted to determine the number of lanes to be used in the design.

Shoulders. Shoulders should be provided with widths as shown in Table 3-11.

Intersections

In the design of intersections, careful consideration should be given to the appearance of the intersection from the driver's perspective. Design should be kept simple to avoid driver confusion. In addition, adequate sight distance should be provided, especially in maneuver or conflict areas. See Stopping Sight Distance and Intersection Sight Distance in Chapter 2, Section 4 for further information regarding sight distance. For guidance on Alternative Intersections and Interchanges, see Appendix E.

Right angle crossings are preferred to skewed crossings. Alignment modifications are generally necessary where skew angles exceed 75 or 105 degrees. Turn Lanes may be provided in accordance with previous discussions in this manual.

Chapter 7, Section 7 Minimum Designs for Truck and Bus Turns provides information regarding the accommodation of various types of truck class vehicles in intersection design. AASHTO's *A Policy on Geometric Design of Highways and Streets* should be consulted for further information on intersection design. Additional information can be found in this manual regarding Sight Distance in Chapter 2, Section 4. Intersections with narrow, depressed median sections, may require superelevation across the entire cross section to provide for safer operation at median openings.

Intersections formed due to route relocation should be designed so as not to mislead drivers. Treatment of an old-new route connection is illustrated in Figure 3-15.

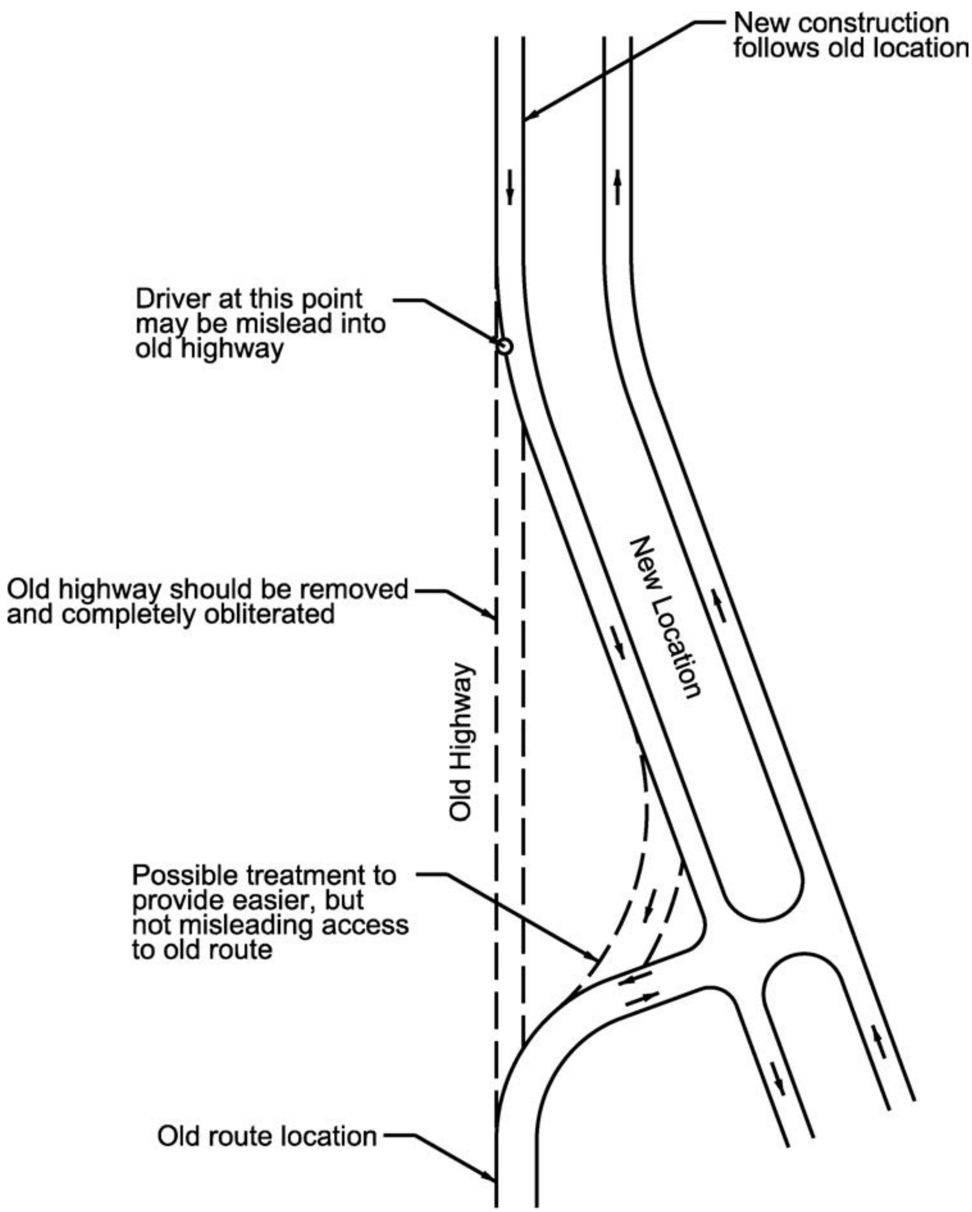


Figure 3-15. Treatment of Old-New Route Connection at Point Where Relocation Begins.

Transitions to Four-Lane Divided Highways

Typical transitions from a two-lane to a four-lane divided highway are shown in Figure 3-16. Transition geometric design criteria is based on the highest design speed of the two roadways. The transition should be visible to the driver approaching from either direction and median openings should not be permitted within one-quarter mile of the transition area. Transition areas should be located so that obstructions such as restrictive width bridges or underpasses or other fixed objects are not within the no-passing zone of the two-lane highway approach.

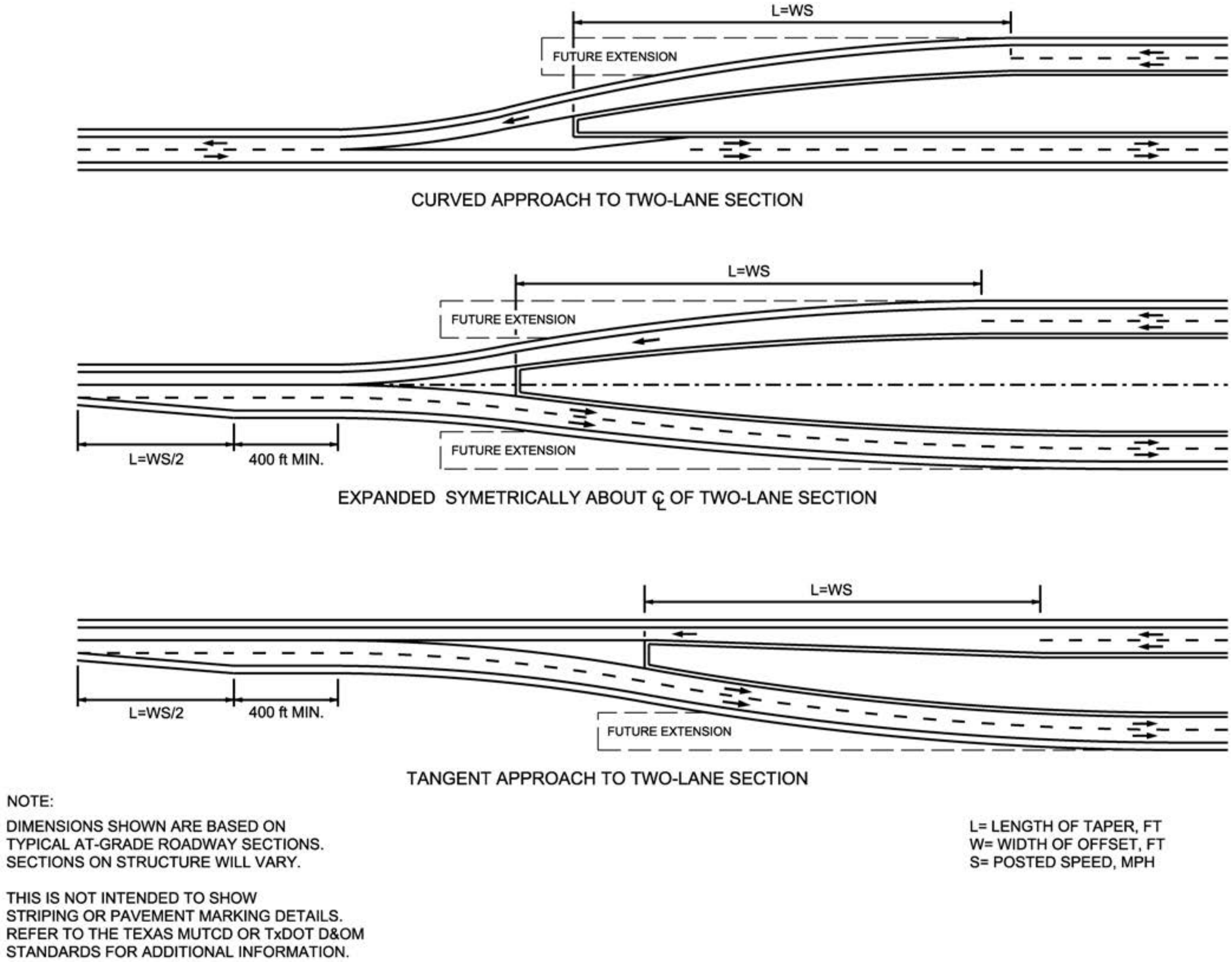


Figure 3-16. Typical Transitions from Two-Lane to Four-Lane Divided Highways.

Converting Existing Two-Lane Roadways to Four-Lane Divided Facilities

When converting an existing two-lane roadway to a four-way divided facility, the Federal Highway Administration (FHWA) allows existing alignments to remain in place. Specifically, the new roadbed will be constructed to full current standards. When the existing lanes are converted to one-way operations, no changes are required to the horizontal or vertical alignment of the existing road as long as it meets 3R criteria (see Chapter 4). Other features such as signing, roadside hardware, and safety end treatments, should meet current standards.

Existing structures with substandard widths on the existing lanes may remain if that width meets minimum rehabilitation (3R) requirements for multi-lane facilities.

Before converting the existing two-lane roadway, a crash analysis of should be conducted to identify specific areas with high crash frequencies so that corrective measures can be taken where appropriate.

Grade Separations and Interchanges

Grade separations or interchanges on multilane rural highways may be provided at high-volume or high-crash rate highway or railroad crossings.

For additional discussion on grade separations and interchanges, see Grade Separations and Interchanges, see Section 6 Freeways and Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*.

Section 6 — Freeways

Overview

Freeways are arterial highways with full control of access. They are intended to provide high levels of safety and efficiency in the movement of large volumes of traffic at high speeds. This section discusses the features and design criteria for freeways and includes the following subsections:

- Basic Design Criteria;
- Access Control;
- Mainlanes;
- Vertical and Clear Zones at Structures;
- Frontage Roads;
- Interchanges;
- Ramps and Direct Connectors;
- Collector-Distributor Roads; and
- Frontage Road Turnarounds and Intersection Approaches.

Basic Design Criteria

Specific references to Freeway Geometric Design criteria are shown in Table 3-15:

Table 3-15: Freeway Geometric Design Criteria

Design Criteria	Reference
General	
Design Speed Mainlanes (urban and rural)	Table 3-17
Design Speed Frontage Roads (urban and rural)	Chapter 3, Frontage Roads
Minimum Horizontal Radius	Table 2-4 and 2-5
Vertical Curvature	Figures 2-6 and 2-7
Maximum Grades (%)	Table 2-9
Stopping Sight Distance	Table 2-1
Vertical Clearance, New Structures	Table 2-11
Superelevation	Chapter 2, Superelevation Rate, Superelevation Transition Length, Supereleveation Transition Placement
Pavement Cross Slope	Chapter 2, Pavement Cross Slope

Table 3-15: Freeway Geometric Design Criteria

Design Criteria	Reference
Lane and Shoulder Widths	Table 3-18
Clear Zone	Table 2-12
Capacity and LOS Analysis	*Highway Capacity Manual*
Turning Radii	See Chapter 7, Minimum Designs for Truck and Bus Turns

Access Control

This subsection discusses access control and includes the following topics:

- General;
- Mainlane Access;
- Frontage Road Access; and
- Control of Access Methods.

General

The entire Interstate Highway System and portions (see TxDOT Statewide Planning Map https://www.txdot.gov/apps/statewide_mapping/statewideplanningmap.html) of the State Highway System have been designated by the Texas Transportation Commission as Controlled Access Highways. It can be necessary to limit or deny an abutting owner's access rights along certain sections of highways, which includes the right of ingress and egress and the right of direct access to and from the owner's abutting property to the highway facility. Such access may be controlled under the State's Police Power, which is an inherent right of sovereignty. However, the existing right of access to an existing public way is an increment of ownership and one of the rights vested in the owner of abutting property. This legal right may be limited or completely denied under the State's police power, which may entitle the owner to potential damages suffered by the loss of such access.

The abutting owners are denied access to any controlled access highway on new location, unless there is a specific grant of access, and no damages may be claimed for the denial of access to the new facility. The theory behind this law is the understanding that the owner cannot be damaged by the loss of something which the owner never had. The denied access or Access Denial Line (ADL) (a.k.a., Control of Access line) is established as part of the property rights acquired by the State for the new location access controlled highway. For ADL established in this manner, if an adjacent property owner requests a change in ADL then the change should be valued under the appropriate Department procedures. The ADL change must be supported by an engineering review to prevent access to that part of the roadway designated as an access controlled highway, as outlined in TxDOT's Access Management Manual.

An owner's rights are taken if an existing road is converted into a controlled access facility, the design of which does not contemplate the initial construction of frontage road(s), and the abutting owner is to be denied access to such facility pending frontage road construction. If an existing road is converted into a controlled access facility, the design of which does contemplate frontage road(s) in the initial construction, and the abutting owner is not to be denied access to such frontage road(s), there is not taking or denial of access rights. Access to the frontage road(s) constitute access to the facility. Further control of movements, once upon the frontage road, such as one-way traffic, no U-turns, no left or right turns, denial of direct access to the through lanes, and circuitous routes are all controlled under police power and inflict no more control over the abutting owner than is inflicted upon the general public.

If an existing road is converted into a controlled access facility and no part of the abutting owner's property is taken for right of way, but access is to be denied to the controlled access facility, and by reason of such denial of access it is found that such owner will suffer damages measured by the diminution of the market value of said abutting land, said owner should be requested to release and relinquish said access rights for consideration equal to the State's approved value for such damages. If the owner is not willing to negotiate on these terms, then the access right may be acquired through eminent domain proceedings. In some instances, the State's appraisal and approved value may indicate that there is no diminution in value by reason of the access denial. In those cases the Department will not proceed with acquisition of the access control and will cancel the A.C. Parcel. There will be no offer of compensation to the owner and the owner will not be required or asked to sign a Release and Relinquishment of Access Rights to a Highway Facility. The ADL will remain on the State Right of Way Map memorializing the denial of access under the Department's police powers and its status will be annotated on the State Right of Way map.

Mainlane Access

Freeway mainlane access, either to or from abutting property or cross streets, is only allowed to occur through a ramp. This control of mainlane access may be achieved through one of the following methods:

- Through access restrictions whereby the access to the highway from abutting property owners is denied with ingress and egress to the mainlanes only at selected freeway or interchange ramps; or
- Through construction of frontage roads permitting access to the mainlanes only at selected ramps.

  In either case, direct access from private property to the mainlanes is prohibited without exception.

Frontage Road Access

Information on the driveway clearance from the cross-street intersection is contained in TxDOT's *Access Management Manual (Chapter 2 Section 3)* and should be considered when locating driveways on projects involving the construction or reconstruction of ramps and/or frontage roads. The remainder of this section addresses driveway and side street access in relation to ramps.

Where frontage roads are provided, access at ramp junctions should be controlled through access restrictions or the use of the State's police powers to maintain operations. The “Control of Access Methods” Section below discusses this further. The placement of streets and driveways near ramp junctions with frontage roads should be carefully considered and permitted only after a traffic operations and safety evaluation.

On reconstruction projects, it may be necessary to close or relocate driveways to meet these guidelines. If the closure/relocation is not feasible, and adjustment of the location of the ramp gore along the frontage road is not feasible, then deviation from these recommended guidelines should be supported by a traffic operations and safety evaluation.

Access Beyond Exit Ramps. Figure 3-17 shows the recommended access control strategy for planned exit ramps and should be used where practical. This figure along with the accompanying values in Table 3-16 show the desirable spacing to be used between exit ramps and driveways, side streets, or cross streets if practical. The number of weaving lanes is defined as the total number of lanes on the frontage road downstream from the ramp. Increased weaving, resulting in operational degradation, occurs when driveway or side street access on the frontage road is in close downstream proximity to exit ramp terminals. For this reason it is important to maintain appropriate separation between the Intersection of Travel Lanes (see Figures 3-17 and 3-18) and downstream driveways or side streets.

It is recognized that there are occasions when meeting the exit ramp separation distance values in Table 3-16 may not be possible due to the nature of the existing development. In these cases, at least 250-ft of separation should be provided from the intersection of the exit ramp and frontage road travel lanes to the downstream driveway or side street. Careful consideration should be given in situations like this, since the minimal separation distance may negatively impact the operation of the frontage road, exit ramp, driveway and/or side street traffic. When a minimum 250-ft separation distance cannot be obtained, consideration should be given to channelization methods that would restrict access to driveways within this 250-ft distance. Refer to the Texas MUTCD for specific types of channelization.

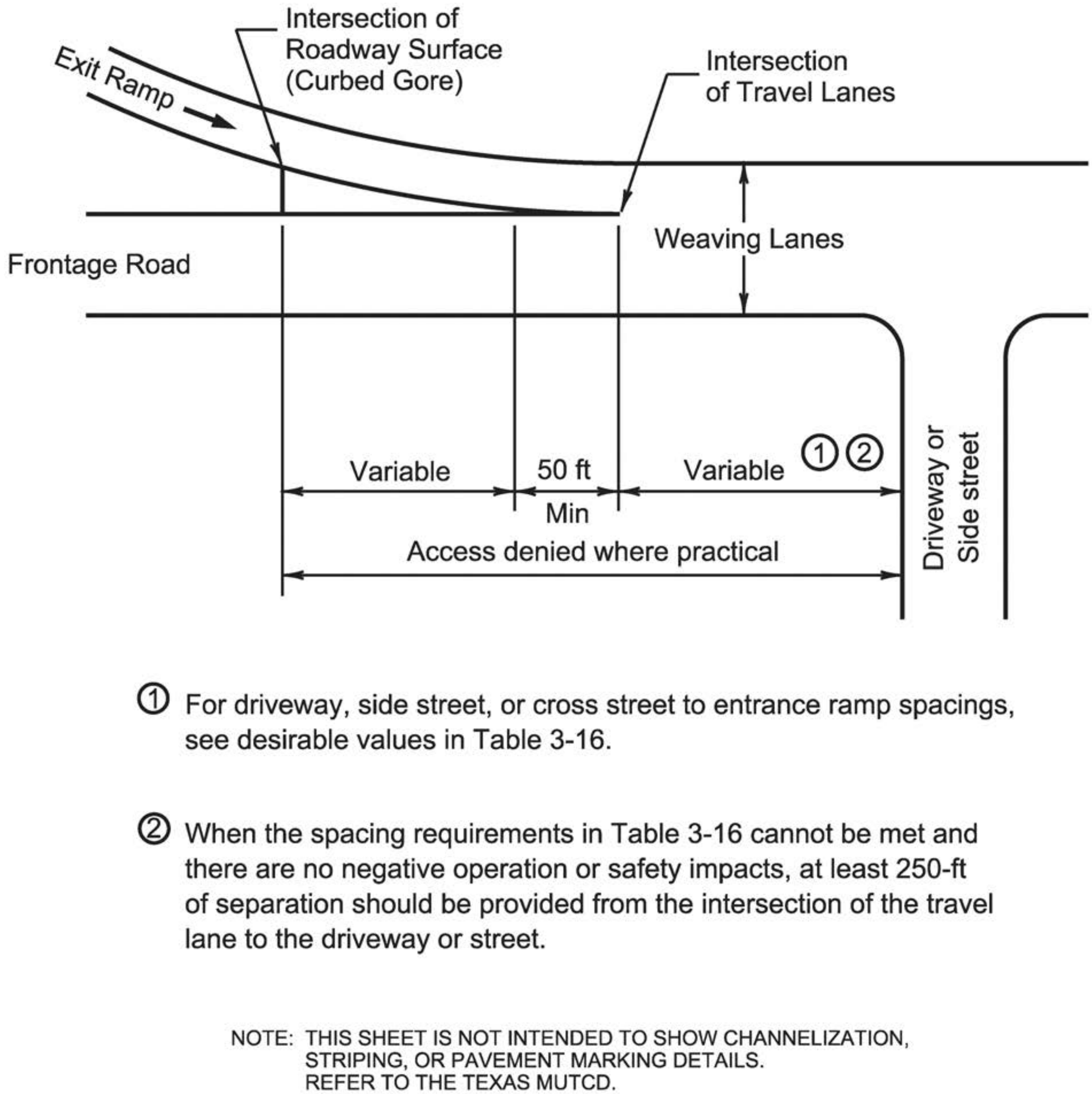


Figure 3-17. Recommended Access Control at Exit Ramp Junction with Frontage Road.

Table 3-16: Desirable Spacing between Exit Ramps and Driveways, Side Streets, or Cross Streets

Total Frontage Road + Ramp Volume (vph)	Driveway or Side Street Volume (vph)	Desirable Spacing (ft)		
		Number of Weaving Lanes on Frontage Road		
		2	3	4
< 2500	< 250	460	460	560
--	> 250	520	460	560
--	> 750	790	460	560

Table 3-16: Desirable Spacing between Exit Ramps and Driveways, Side Streets, or Cross Streets

Total Frontage Road + Ramp Volume (vph)	Driveway or Side Street Volume (vph)	Desirable Spacing (ft)		
		Number of Weaving Lanes on Frontage Road		
		2	3	4
--	> 1000	1000	460	560
> 2500	< 250	920	460	560
--	> 250	950	460	560
--	> 750	1000	600	690
--	> 1000	1000	1000	1000

Access Prior to Entrance Ramps. The presence of driveways or side streets in close upstream proximity to entrance ramp terminals, similar to exit ramps, increases weaving and results in operational degradation of the frontage roads. Therefore, maintaining proper separation is important. Figure 3-18 shows the recommended access control strategy for entrance ramps and should be used where practical. There will be occasions when meeting the entrance ramp separation distance values shown in Figure 3-18 may not be possible due to existing development conditions. In these cases, at least 100-ft of separation distance should be provided between the intersection of the entrance ramp and frontage road travel lanes and the upstream driveway or side street.

This limited separation negatively impacts the operation of the frontage road, entrance ramp, driveway, and/or side street traffic, therefore, careful consideration should be given to its use. When the 100-ft separation distance for entrance ramps cannot be obtained, consideration should be given to channelization methods that would restrict access to driveways within this 100-ft distance. Refer to the Texas MUTCD for specific types of channelization.

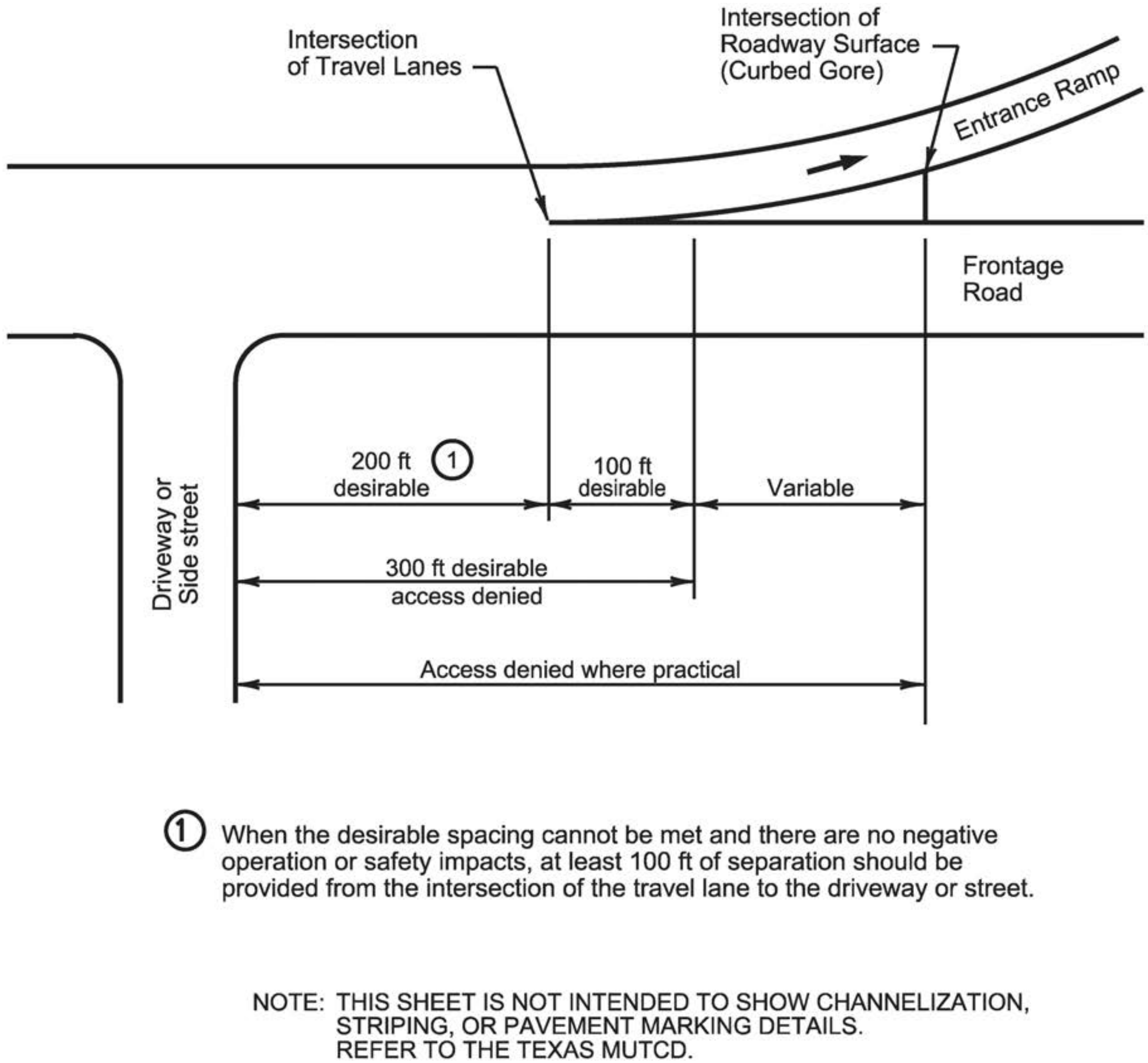


Figure 3-18. Recommended Access Control at Entrance Ramp Junction with Frontage Road.

Ramp Location. During schematic development, care should be exercised to develop design in sufficient detail to accurately tie down the locations of ramp junctions with frontage roads and the location of access control limits; reference Chapter 1, Section 3 for specific guidance on schematic development. These drawings are often displayed at meetings and hearings and become the basis for right-of-way instruments or the Department's regulation of driveway location for that project. Figures 3-17 and 3-18 provide recommended access control at exit and entrance ramp junctions with frontage roads.

In some instances, ramps must be shifted to satisfy level of service considerations or geometric design controls. When this is necessary, the access control limits should also be shifted if right-of-way has not been previously purchased.

Direct Access to a Ramp. The following requirements apply to Direct Access to a ramp from an adjacent property or street:

- All ramps with a frontage road: Direct access to a ramp is always prohibited for the full length of the ramp.
- Interstate freeway ramp without a frontage road or an Interstate interchange connector: Direct access is always prohibited under 23 CFR 625.3 and 625.4 for the full length of the ramp or Interstate interchange connector.
- Non-Interstate facility without a frontage road, with or without access controlled by designation: Direct access is strongly discouraged. If allowed, the access location must be determined through the procedures and spacing criteria contained in this chapter, and in the TxDOT *Access Management Manual*.

Control of Access Methods

A controlled access highway may be developed in either of two ways:

- Designation (Transportation Code §203.031 Control of Access); and
- Design (continuous frontage road and State's police power) See TxDOT's *Access Management Manual* for additional information.

Control of Access by Designation. When the Texas Transportation Commission designates a freeway to be developed as a controlled access facility under Transportation Code §203.031, the State is empowered to control access through access restrictions. All Interstate Highways are designated as controlled access and certain other routes have been or may be designated. These designated freeways may or may not have frontage roads, whichever arrangement is determined to be appropriate as discussed in Planning Development of freeways by designation, rather than solely by design, is the preferred design approach especially for all new location freeways.

Not Along an Existing Public Road. Per *Transportation Code §203.031*, wherever designated controlled access freeways include frontage roads and the planned location is not along an existing public road, access should be controlled through access restrictions at ramp junctions with frontage roads as shown on Figure 3-17 and Figure 3-18.

Where no frontage roads are provided, access is controlled to the mainlanes by access restriction.

Along an Existing Public Road. Per *Transportation Code §203.031*, whenever a designated controlled access freeway is to be provided along an existing public road, frontage roads are generally provided to retain or restore existing access. This is subject to the guidelines presented in the Planning section of this document.

Frontage road access should be controlled by imposing recommended access restrictions in accordance with Figure 3-17 and Figure 3-18 whenever all of the following conditions prevail:

- Right-of-way is being obtained from the abutting property owner(s);
- A land locking condition does not result;

Access may be controlled by use of the State's police power to control driveway location and design where any of the following conditions prevail:

- No right of way is obtained from the abutting property owner(s); or
- Restricting access results in landlocking an abutting property.

Whenever the State's police powers are used, the denial of access zone should be free of driveways insofar as practical.

(Some designated controlled access freeways may be a combination of new location and along an existing road. Both Figure 3-17 and Figure 3-18 and police powers may be used at appropriate locations)

Control of Access by Design

If an existing highway is to be developed as a controlled access facility solely by design (not designated by the Transportation Commission), TxDOT is not empowered to purchase access rights. Instead, TxDOT must achieve access control by construction of continuous frontage roads and by the utilization of the State's police power to control driveways, particularly at ramp junctions with frontage roads.

In the interest of providing for highway safety and utility, the State may regulate driveway location and design through its police powers. Landlocking through complete denial of access is beyond the State's regulatory power (without Commission designation under the Transportation Code). The State, however, may effectively regulate driveway location in accordance with statewide policy as long as the following two conditions are met:

- Reasonable access is provided, and
- Land locking of an abutting property does not result.

The TxDOT *Access Management Manual* governs design and location of driveways.

The design philosophy from the section on Frontage Roads applies whenever new or relocated ramps are to be provided along existing freeways. Access should be controlled at frontage road junctions through access restriction as illustrated in Figure 3-17 and Figure 3-18.

Whenever access is to be controlled solely by provision of frontage roads, State police power to regulate driveway location and design should be used to control access near ramp junctions. However, where designation by the Transportation Commission is practical, use of access restrictions as illustrated in Figure 3-17 and Figure 3-18 is preferred over controlling access solely by design (police power).

Mainlanes

This subsection discusses mainlanes and includes information on the following topics:

- Design Speed;
- Level of Service;
- Lane Width and Number;
- Shoulders;
- Medians;
- Outer Separation; and
- Crossing Facilities.

Design Speed

The design speed of urban freeways should reflect the anticipated operating conditions during non-peak hours. However, the design speed should not exceed the limits of prudent construction, right-of-way, and socioeconomic costs. Minimum design speeds for rural and urban freeways are shown in Table 3-17.

Table 3-17: Design Speed for Controlled Access Facilities

Facility	Minimum (mph)
Mainlanes - Urban	50
Mainlanes - Suburban	60
Mainlanes - Rural	70

Level of Service

For acceptable degrees of congestion, urban freeways and their auxiliary facilities should generally be designed for Level of Service (LOS) C, as defined in the *Highway Capacity Manual*, in the design year. In heavily developed urban areas, LOS D may be acceptable. In heavily congested areas, other Measure of Effectiveness (MOEs) may include travel time, speed and queue lengths.

In rural areas, LOS B is desirable for freeway facilities; however, LOS C may be acceptable for auxiliary facilities (i.e., ramps, direct connections and frontage roads) carrying unusually high volumes.

Lane Width and Number

The minimum and usual mainlane width is 12-ft. The number of lanes required to accommodate the anticipated traffic in the design year is determined by the level of service evaluation as discussed in the *Highway Capacity Manual*. See Table 3-18 and Figure 3-20 for further information.

After the number of lanes has been determined, the balance in the number of lanes should be confirmed based on the following principles:

1. At entrances, the number of lanes beyond the merging of two traffic streams should not be less than the sum of all traffic lanes on the merging roadways minus one but may be equal to the sum of all traffic lanes on the merging roadways.
2. At exits, the number of approach lanes on the highway should be equal to the number of lanes on the highway beyond the exit, plus the number of lanes on the exit, minus one. Exceptions to this principle occur at clover leaf loop-ramp exits that follow a loop-ramp entrance and at exits between closely spaced interchanges. (Closely spaced interchanges are those 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 1,500-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 such that the number of lanes on the approach roadway is equal to the number of through lanes beyond the exit plus the lane on the exit.
3. The traveled way of the highway should be reduced by not more than one traffic lane at a time.
4. To satisfy lane-balance at two-lane exits, an auxiliary lane upstream of the exit should be provided.
5. To satisfy lane-balance at two-lane entrances, at least one additional lane should be provided downstream. This lane may be another through lane if needed for capacity or an auxiliary lane that may be reduced with the appropriate taper or at the next interchange.

Typical examples of lane balance are shown in Figure 3-19.

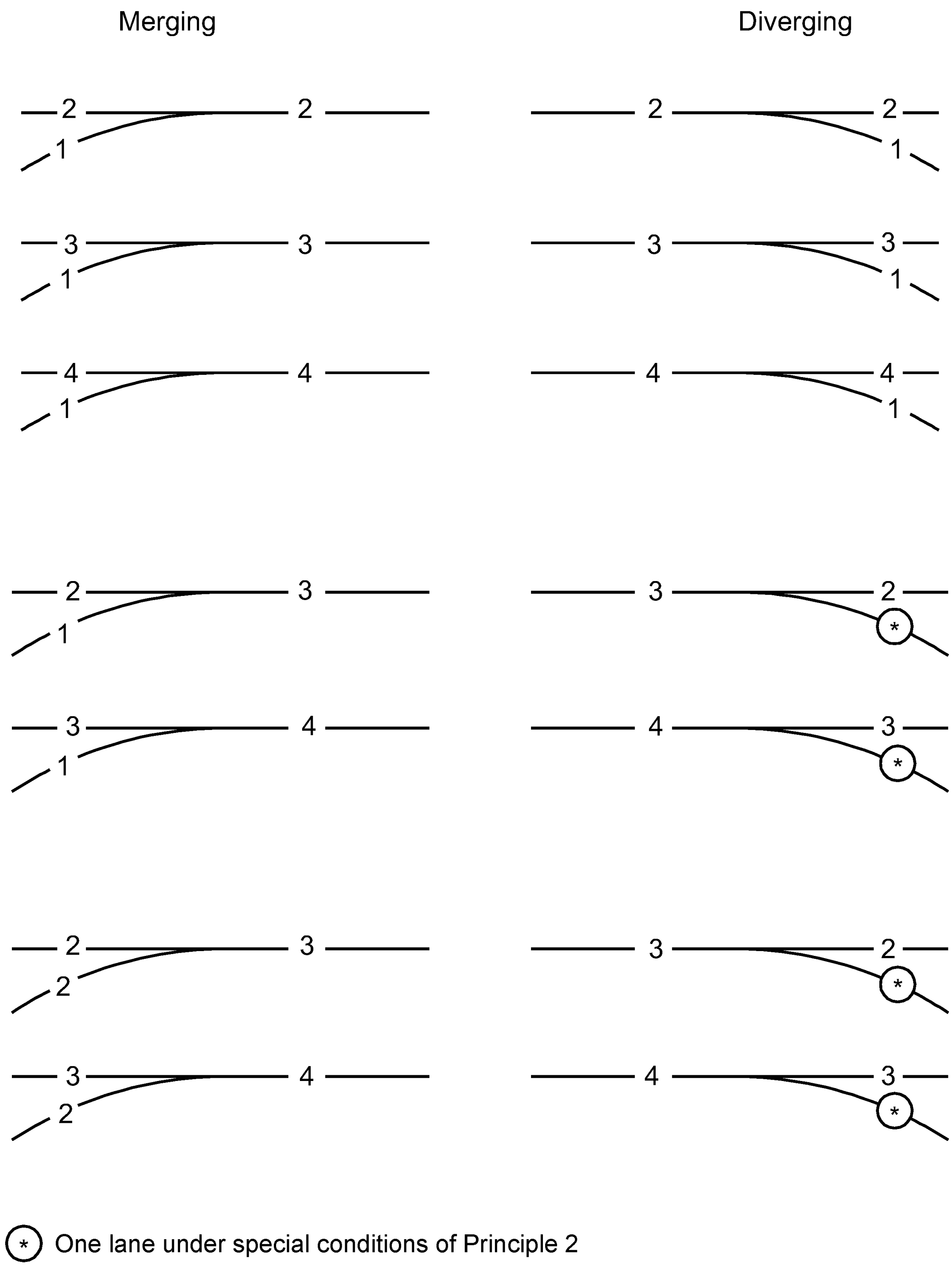
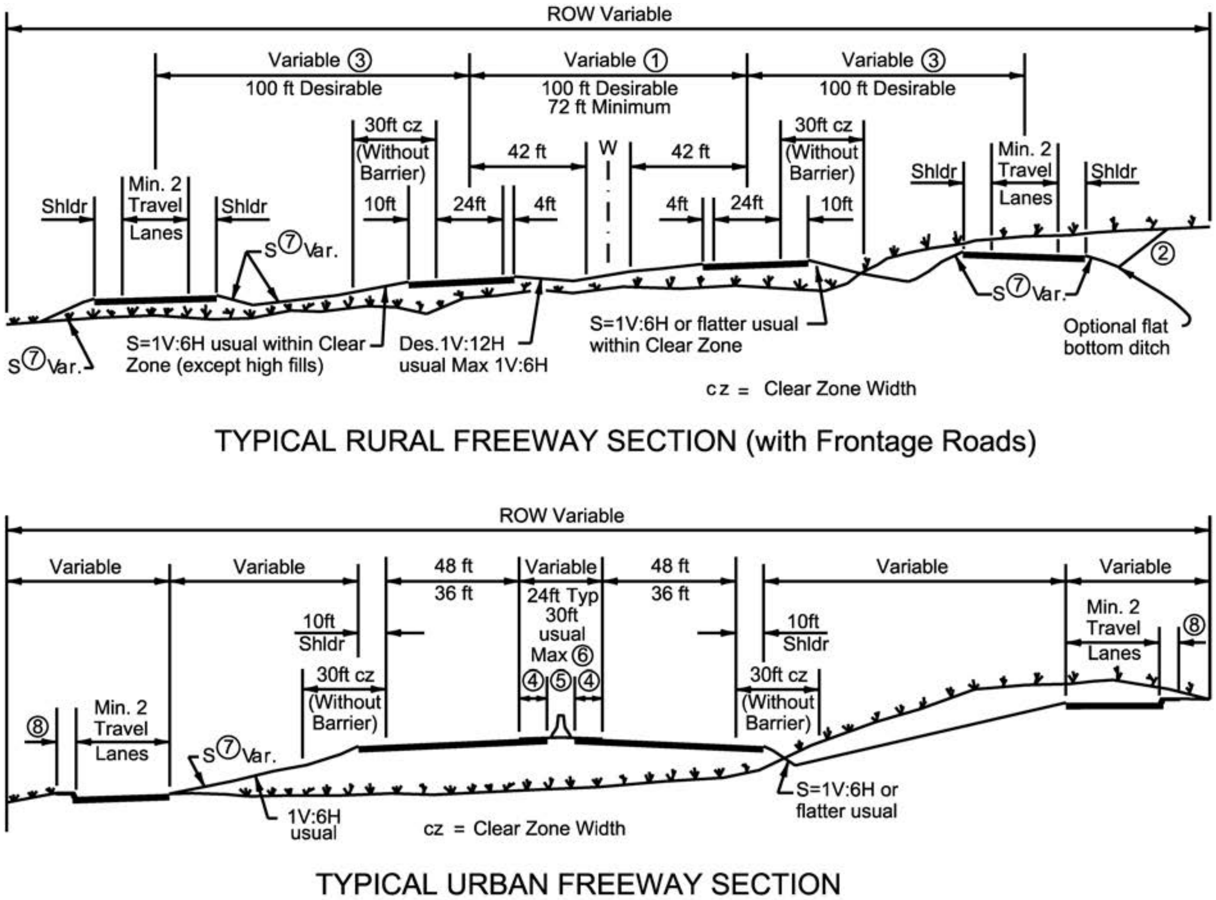


Figure 3-19. Typical Examples of Lane Balance.

See AASHTO's *A Policy on Geometric Design of Highways and Streets* for design details.

See the latest version of the Texas MUTCD, Freeway Pavement Marking Standard and the TxDOT Freeway Signing Handbook for additional guidance on signing and pavement marking design.



① For minimum, 30 ft clearance to obstruction in medians. Width center line to center line of 84 ft + W is required. W = width of obstruction.

② Backslope in cuts may be exceeded in rock.

③ Additional width required in interchange areas.

④ 10 ft minimum on six lanes.

⑤ 4-ft minimum and usual. Median barrier generally used only in medians of 30 ft usual max or less.

⑥ A 48 ft median is appropriate where a future additional lane in each direction is planned.

⑦ See Chapter 2, Section 6 for discussion on slope rates.

⑧ Sidewalk or shared-use path (SUP), if applicable.

Figure 3-20. Typical Freeway Sections.

Shoulders

Continuous surfaced shoulders are provided on each side of the mainlane roadways, both rural and urban, as shown in Figure 3-20. For four-lane freeways, the minimum shoulder widths must be 10-ft on the outside and 4-ft on the inside (median). On freeways of six lanes or more, 10-ft inside shoulders must be provided for emergency pull-offs. A 10-ft outside shoulder must be maintained along all speed change lanes. However, in weaving areas with LOS A or LOS B, a 6-ft shoulder can be considered. See Table 3-18 and Figure 3-20 for further information. For shoulders that are designated as emergency evacuation lanes, a 12-ft outside shoulder is recommended.

Table 3-18: Roadway Widths for Controlled Access Facilities

<table>
<tr><th>Type of Roadway</th><th>Inside Shoulder Width (ft.)</th><th colspan="2">Outside Shoulder Width (ft.)</th><th>Traffic Lanes (ft.)</th></tr>
<tr><td colspan="5">Mainlanes:</td></tr>
<tr><td>4-Lane Divided</td><td>4</td><td colspan="2">10</td><td>24</td></tr>
<tr><td>6-Lane or more Divided</td><td>10</td><td colspan="2">10</td><td>36[1]</td></tr>
<tr><td>1-Lane Direct Conn.[2]</td><td>2 or 4[4] Rdwy.; 4[4] Str.</td><td colspan="2">8</td><td>14</td></tr>
<tr><td>2-Lane Direct Conn.[2]</td><td>2 or 4[4] Rdwy.; 4[4] Str.</td><td colspan="2">8</td><td>24</td></tr>
<tr><td colspan="5">Ramps:</td></tr>
<tr><td colspan="2"></td><td>Minimum</td><td>Desirable</td><td></td></tr>
<tr><td>Ramps[2] (uncurbed)</td><td>2 or 4[4] Rdwy.; 4[4] Str.</td><td>6</td><td>8</td><td>14[5]</td></tr>
<tr><td>Ramps[3] (curbed)</td><td>-</td><td>-</td><td>-</td><td>22[5]</td></tr>
<tr><td>Speed Change Lanes</td><td>-</td><td>6</td><td>10</td><td>12</td></tr>
<tr><td colspan="5">Notes:
1. For more than six lanes, add 12-ft width per lane.
2. If sight distance restrictions are present due to horizontal curvature, swapping the widths of the inside and outside shoulders is allowed (i.e. the shoulder width on the inside of the curve may be increased to 8-ft and the shoulder width on the outside of the curve decreased to 2-ft (Rdwy) or 4-ft (Str)) but can be no less than the values stated in this footnote (see Note 4).
3. The curb for a ramp lane will be mountable and limited to 4-inches or less in height. The width of the curbed ramp lane is measured face to face of curb. Existing curb ramp lane widths of 19-ft may be retained.
4. Roadways with longitudinal traffic barriers must provide a minimum 4-ft shoulder from the travel lane edge. The shoulder across a bridge will be measured from the travel lane edge to the nominal face of rail. This provides additional offset for high-speed operation and door-opening space for vehicles stopped on the shoulder of the roadway.
5. 12-ft lanes are permissible on ramps with multiple lanes.</td></tr>
</table>

Medians

The width of the median is the distance between the inside edges of the travel lanes. For depressed freeway sections, 76-ft medians are generally used. Where topography, right-of-way, or other special considerations dictate, depressed freeway median width may be reduced from 76-ft to a minimum of 48-ft. A median width of 24-ft to 30-ft is generally used on freeway sections with flush medians. On freeways including six or more travel lanes and a flush 24-ft median, the resulting section provides for 10-ft inside shoulders and a usual 2-ft offset to barrier centerline. See Figure 3-20 for further information.

Design the ultimate freeway section initially where feasible to reduce the adverse impact to high-speed and high-volume traffic resulting from construction in multiple phases. Several benefits could be realized from initially constructing the ultimate section such as lower levels of traffic handling and traffic shifts, minimal temporary pavement and interim transitions, construction and inflation cost savings, and plans to accommodate future bridge columns and other appurtenances. Under those circumstances where future additional lanes will be provided in the median area, the usual median width of 24-ft should be increased by the appropriate multiple of 12-ft in anticipation of need for additional lanes. Provisions should be made, or retained, for any future high occupancy vehicle lanes in the median.

On freeways where median barrier is used, the horizontal alignment must be checked for adherence to the stopping sight distance criteria accounting for any restriction caused by the barrier.

For information on freeway median crossings, refer to Chapter 7, Section 6 Emergency Median Crossovers.

Outer Separation

The portion of the freeway between the mainlanes and frontage road or right-of-way, where frontage roads are not provided, should be wide enough to accommodate shoulders, speed change lanes, side slopes and drainage, retaining walls and ramps, and signs and other appurtenances necessary for traffic control. Because of right-of-way limitations in urban areas, the outer separation may oftentimes be narrower than desired; however, in rural areas, where opposing headlights along a two-way frontage road tend to reduce a driver's comfort and perception on the freeway, the outer separation should be as wide as possible. Clear zones, as defined in Chapter 2, are desirable for errant vehicle recovery and safety.

Crossing Facilities

The following references show the appropriate widths for facilities crossing the freeway:

- Urban Streets: Table 3-1;
- Suburban Roadways: Table 3-5;
- Two-Lane Rural Highways: Table 3-6 and Table 3-8; and
- Multilane Rural Highways: Table 3-11;

The Bridge Project Development *Manual* should also be referenced for appropriate structure widths.

Vertical and Clear Zones at Structures

Vertical. All controlled access highway grade separation structures, including railroad underpasses, should provide the minimum vertical clearance over the usable roadway shown in Table 2-11. Specific railroad company guidelines may also require additional vertical clearance requirements as a function of structure type. This information can be found in the applicable railroad company guidelines. Additionally, all freeways are part of the Texas Highway Freight Network (THFN), and therefore may require additional THFN vertical clearance requirements as stated in Chapter 2, Section 6 and Chapter 3, Section 8.

Structures over the mainlanes of Interstate or controlled access highways must meet the minimum vertical clearance requirement in Table 2-11 except within cities where the vertical clearance is provided on an Interstate loop around the particular city. See Chapter 1, Section 2 for specific guidance and coordination for design exceptions for Interstates.

Roadways under the mainlanes of interstate or controlled access highways must meet the minimum vertical clearance requirements for the appropriate undercrossing roadway classification.

Minimum vertical clearances for pedestrian crossover structures are specified in Table 2-11. This is due to the increased risk of personal injury upon impact by over-height loads and the relative weakness of such structures to resist lateral loads from vehicular impact.

The clearances specified above apply to the entire width of roadway mainlanes including usable shoulders and include an allowance of 6-inches for future pavement overlays. It is recognized that it is impractical to arrive at the exact clearance dimensions on the structure plans.

Vertical clearance for railroad overpasses is discussed in the *Bridge Project Development Manual.*

Horizontal. For the minimum clear zone to bridge parapets and piers see Table 2-12.

Frontage Roads

This subsection discusses frontage roads and includes information on the following topics:

- Function and Uses;
- Planning;
- Design Speed on Frontage Roads; and
- Capacity and Level of Service.

Function and Uses

Frontage roads serve a multitude of purposes in addition to controlling or providing access. In urban settings, frontage roads reduce the “barrier” effect of urban freeways by providing for some of the circulation of the local street. They provide invaluable operational flexibility, serving as detour routes during mainlane crashes, mainlane maintenance activity, for over-height loads, as bus routes, and during inclement weather. Continuous frontage roads provide the operational flexibility required to manage saturation for freeways that include freeway surveillance and control.

In addition to these functions, frontage roads can prove advantageous when used as the first stage of construction for a future freeway facility. By constructing frontage roads prior to the mainlanes, interim traffic demands can often be satisfied and a usable section of highway can be opened to the traveling public at a greatly reduced cost.

Planning

Frontage roads may be incorporated into a project at various points during the project development, including the planning stage, subsequent to the planning stage, or after freeway construction. However, later incorporation of frontage roads will be more difficult.

Frontage road construction may be funded by TxDOT, a local government, or shared by both. The Texas Transportation Commission has adopted rules governing the construction and funding of frontage roads. All frontage road development must be in accordance with the rules contained in 43 Texas Administrative Code (TAC) §15.54. TxDOT's *Project Development Process Manual* can also be referenced for additional information.

Changes in control of access must be in accordance with 43 TAC §15.54(d)(4).

Subsequent changes in the control of access will be as shown on approved construction plans or as provided in instruments conveying right-of-way on authorized projects, or as authorized by Commission Minute Order, and specified in TxDOT's *ROW Preliminary Procedures for the Authority to Proceed*. Where access is permitted to adjacent properties and the adjacent property owners approach the State for a change in access, ingress and egress will be governed by the issuance of permits to construct access driveway facilities. This process is shown in Department established policy, guidelines and procedures, designed to provide reasonable access, in accordance with the Texas laws.

Design Speed on Frontage Roads

Design speeds for frontage roads are a factor in the design of the roadway. For consistency, design speeds that match values for collector streets or highways should be used. The desirable design speed for urban frontage roads is 50 mph, and the minimum design speed is 30 mph. See Table 3-5 for design speeds for suburban frontage roads (suburban collector criteria), and Table 3-19 for rural frontage roads (rural collector criteria).

Capacity and Level of Service

Although techniques to estimate capacity and level of service on freeways and urban arterials are detailed in the *Highway Capacity Manual*, these procedures should not be applied directly to frontage roads, as frontage roads have characteristic of both freeways (i.e., exit and entrance ramps) and urban arterials (i.e., driveways, cross streets and signalized intersections). The following report was developed to suggest techniques for estimating capacity and level of service on frontage roads.

Kay Fitzpatrick, R. Lewis Nowlin, and Angelia H. Parham. *Procedures to Determine Frontage Road Level of Service and Ramp Spacing*. Research Report 1393-4F, Texas Department of Transportation, Texas Transportation Institute, 1996.

Additional information can be found in TTI Research Report 1393-4F, which contains procedures for the following:

- Determining level of service on a continuous frontage road section;
- Analyzing frontage road weaving sections; and
- Determining spacing requirements for ramp junctions.

Frontage Road Design Criteria

A frontage road should be designed to provide one-way operation. There may be exceptions in certain isolated instances, where a one-way pattern would impose severe restrictions on circulation within an area. These exceptions must be approved by the Design Division at the schematic stage.

Frontage road design criteria can be found in the following tables:

- Urban: Table 3-1;
- Suburban: Table 3-5;
- Rural: Table 3-19; and
- Clear Zone: Table 2-12

Table 3-19: Design Criteria for Rural Frontage Roads

Design Speed[2] (mph)	Min. Width[1] for Future Traffic Volume of			
	0-400 ADT	400-1,500 ADT	1,500-2,000 ADT	2,000 or more ADT
LANES (ft.)				
20	10	10	11	12
25	10	10	11	12
30	10	10	11	12
35	10	10	11	12
40	10	11	11	12

Table 3-19: Design Criteria for Rural Frontage Roads

Design Speed[2] (mph)	Min. Width[1] for Future Traffic Volume of			
	0-400 ADT	400-1,500 ADT	1,500-2,000 ADT	2,000 or more ADT
45	10	11	11	12
50	10	11	12	12
55	10	11	12	12
60	11	11	12	12
65	11	11	12	12
70	11	11	12	12
75	11	12	12	12
80	11	12	12	12
SHOULDERS (ft)[4,5]				
Each Shoulder Two-Way Operation	2[3]	4	8	8 – 10
Inside Shoulder One-Way Operation	2[3]	2[3]	4	4[4]
Outside Shoulder One-Way Operation	2[3]	4	8	8 – 10

Notes:
1. May retain existing paved width on a reconstruction project if total paved width is 24 ft. and operating satisfactorily
2. Use rural collector criteria (Table 3-7) for determining minimum design speed
3. At locations where roadside barriers are provided, use minimum 4-ft. offset from travel lane edge to barrier face
4. If the one-way frontage road section contains three or more travel lanes, then minimum inside shoulder width is 8-ft to 10-ft.
5. A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated Off-system Bridges, with current ADT greater than 400 ADT, where this addition may represent an unreasonable increase in cost may be excepted from the bicycle clear space requirement. See Ch. 6 Section 1 for specific off-system bridge requirements for current ADT of 400 or less.

Conversion of Frontage Roads from Two-Way to One-Way Operation

In some areas, existing frontage roads are operating as two-way facilities. Such two-way operation has the following disadvantages:

- Higher crash rates due to the risk of head-on collisions at the ramp terminals;
- Increased potential for wrong-way entry to the mainlanes;
- Complicated intersections requiring left turns from the arterial onto the frontage road from both directions;
- Restriction from using signal phasing and sequencing options normally available at signalized diamond interchanges; and

- Limited capacity when compared to one-way operations.

Existing two-way frontage roads should be converted to one-way operation when one or more of the following conditions exist:

- Queuing on the frontage road approach that routinely backs up from the arterial intersection to within 100-ft of a freeway entrance or exit ramp gore;
- The Level of Service (LOS) of a signalized intersection of the frontage road and the arterial drop below LOS C;
- Queuing in the counter-flow direction, which would not exist if the frontage road were one-way, routinely backs up from the stop line at a freeway entrance or exit ramp to within 100-ft of the arterial street;
- Crash rates are above the statewide average crash rate for two-way frontage roads; or
- Major freeway reconstruction or rehabilitation is occurring in a developed or developing area.

Conversion of two-way frontage roads in urbanizing rural areas, with relatively long distances between crossover interchanges, will require consideration of additional crossovers to minimize the distance traveled for adjacent residents and business patrons. Existing local street systems in the area will facilitate traffic circulation and minimize the travel time impact of converting frontage roads from two-way to one-way operation.

The simple conversion of two-way to one-way frontage roads will be accomplished with ramp and terminal design based on reconstruction criteria shown in Chapter 3, Section 6 Ramps and Direct Connectors. Other existing frontage road lanes may retain dimensions that meet rehabilitation criteria shown in Chapter 4, Section 4 Frontage Roads. However, if the frontage roads are being reconstructed, then reconstruction design criteria shown in Chapter 3, Section 6 Freeways will be applicable throughout the section.

Major reconstruction or rehabilitation of existing two-way frontage roads shall require approval from the Design Division if the two-way operation is to remain. Any new frontage road must be designed and constructed for one-way operation unless prior approval from Design Division is obtained. Two-way frontage roads should only be considered in rural areas where:

- The distances between crossover interchanges are relatively long;
- The adjoining road system is typically discontinuous;
- The corridor is sparsely developed; and
- Development is not anticipated in the near future.

Grade Separations and Interchanges

The ability to accommodate high volumes of traffic safely and efficiently through intersections depends largely on the arrangement of the intersection. Grade separations offer the greatest efficiency, safety, and capacity for intersecting roadways. A grade separation refers to the crossing of two roadways by a physical separation so that neither roadway interferes with the other. 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 or highways on different levels.

The decision to provide a fully access-controlled facility becomes the warrant for providing highway grade separations or interchanges at the most important intersecting roadways (usually arterials and some collectors) and railroads. No at-grade intersections are allowed along the mainlanes of a freeway.

Effect on Community. An interchange or series of interchanges on a freeway through a community may affect large continuous areas or even the entire community. For this reason, interchanges must be located and designed so that they will provide the best possible traffic service. Drivers who have exited from a freeway expect to be able to re-enter in the same vicinity; therefore, partial interchanges that do not serve all desired traffic movements should be avoided.

Classifications. Interchanges are generally classified according to the number of approach roadways or intersection legs, as three-leg, four-leg and multi-leg interchanges. Three-leg interchanges are commonly referred to as Tee (Trumpet) interchanges. Four-leg interchanges are often referred to as Cloverleaf or Diamond interchanges. Directional interchanges have three or more legs including direct connectors.

The following subsections include a brief description and some of the advantages and disadvantages of each of the types of interchanges.

For information on the design of Diverging Diamond Interchanges, refer to Appendix E.

Three-Leg Interchanges

Three-leg interchanges provide connection for three intersecting approach roadways. These interchanges should only be used after careful consideration because expansion to include a fourth leg is usually very difficult. If the potential exists that a fourth leg will ultimately be included, another type of interchange may be more appropriate.

Trumpet. The most widely used 3-leg interchange is the trumpet, as shown in Figure 3-21. This type of interchange is particularly suitable for the connection of a major facility and a freeway. Preference should be given to the major turning movements so that the directional ramps carry the higher traffic volume and the lower traffic volumes use the loop ramps.

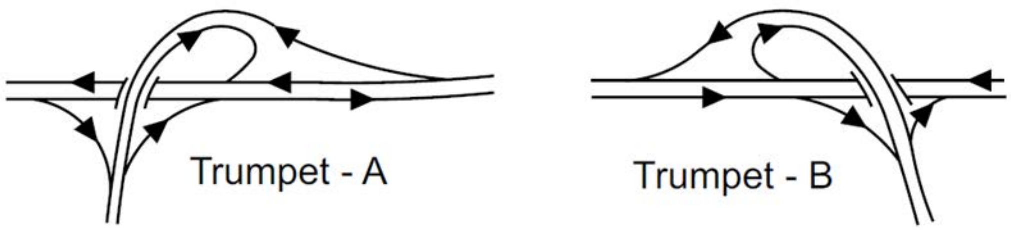


Figure 3-21. Trumpet Three Leg Interchange.

Direct. Direct three-leg interchanges are those in which all movements are provided without the use of loops. These interchanges should be used only where all movements have high traffic volumes. Direct three-leg interchanges require more than one structure or, alternatively, a three-level structure. Both variations are illustrated in Figure 3-22.

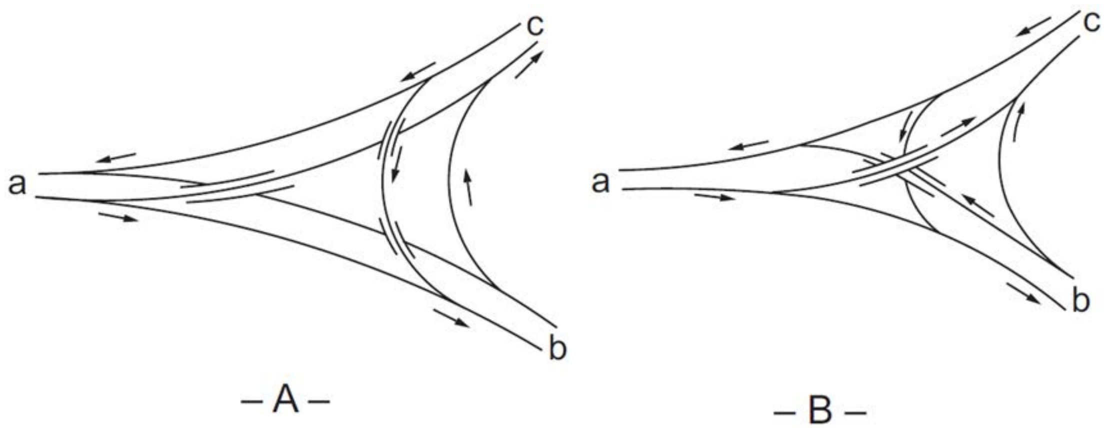


Figure 3-22. Directional Three-Leg Interchange.

Four-Leg Interchanges

Four-leg interchanges can take a wide variety of forms. The choice of interchange is generally established after careful consideration of dominant traffic patterns and volumes, right-of-way requirements, and system considerations. The three primary types of four-leg interchanges are Diamond, Cloverleaf, and Directional.

Diamond Interchanges

The diamond interchange is the most common interchange, especially in urban areas, since it requires less area than any other type. The diamond interchange is used almost exclusively for major-minor crossings since left-turn movements are made at-grade across conflicting traffic on the minor road. Separation between frontage road intersections in diamond interchanges in urban or suburban conditions should desirably be 300-ft as shown in Figure 3-23. Provide proper traffic signal timing if the 300-ft cannot be met.

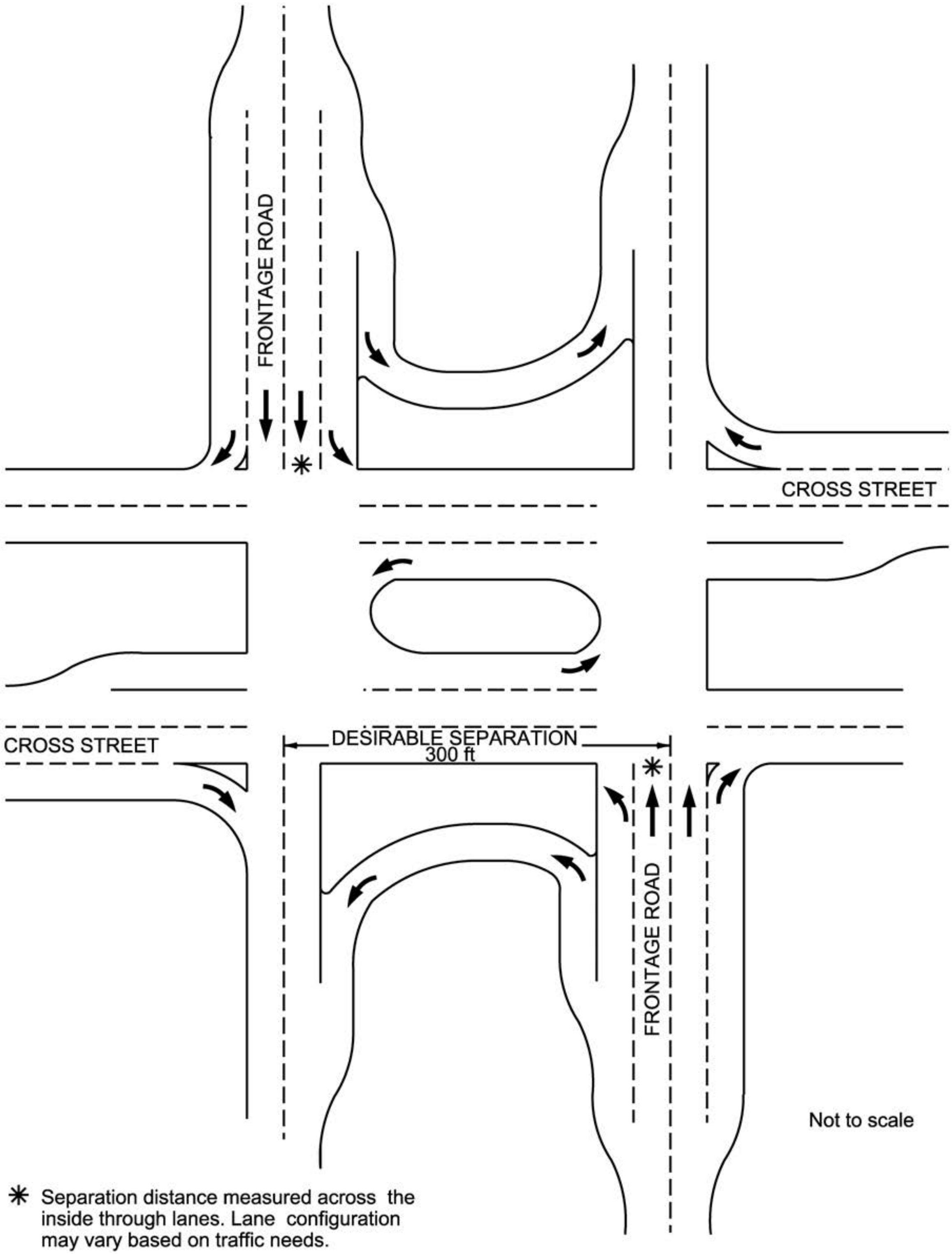


Figure 3-23. Typical Interchange for At-Grade Portion of Diamond Interchange in Urban or Suburban Areas.

The diamond interchange may have several configurations, as discussed in the following paragraphs and shown in Figure 3-24:

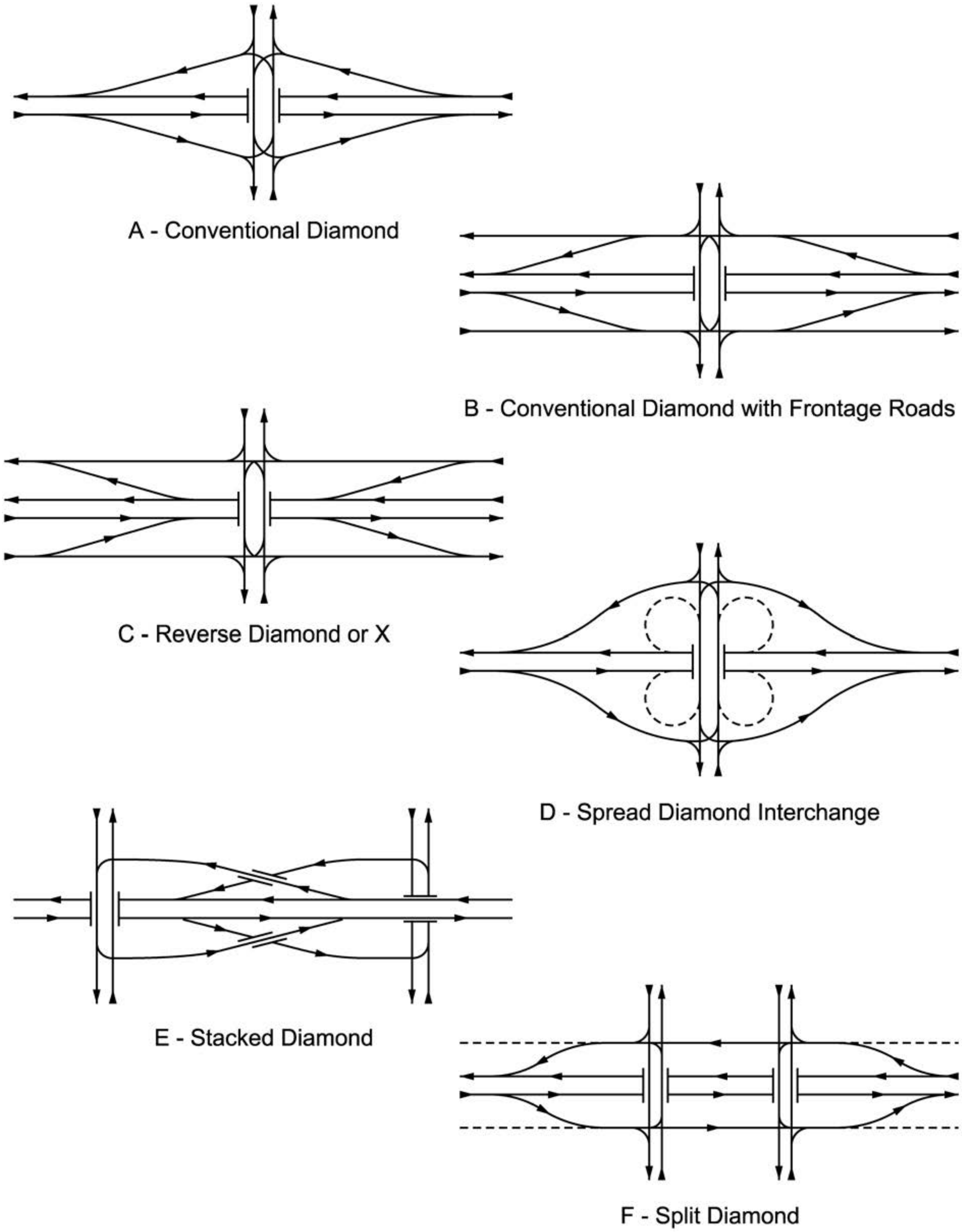


Figure 3-24. Common Diamond Interchanges.

Conventional diamond without frontage roads. The conventional diamond (Figure 3-24(A)) is the most common application of a diamond interchange. Traffic exits in advance of and close to the cross street. Entering vehicles quickly access the freeway just past the cross street. This design can result in vehicles backing up onto the freeway when long queues form on the ramp.

Conventional diamond with frontage roads. The conventional diamond with frontage roads (Figure 3-24(B)) is a common variation of a diamond interchange. Traffic exits in advance of and near the cross street. Entering vehicles quickly access the freeway past the cross street. However, exiting vehicles can sometimes back up onto the freeway when long queues form on the ramp or frontage road, and most vehicles must go through the intersection to gain access to most frontage road property.

Reverse diamond or X- pattern. The reverse diamond or "X" interchange pattern (Figure 3-24(C)) is used in locations with significant development along the frontage road. It provides access between interchanges and prevents exiting vehicles from backing up onto the freeway. However, entering vehicles may have to accelerate on an upgrade and exiting maneuvers occur just beyond the crest vertical curve where weaving also takes place. The "X" ramp pattern also encourages frontage road traffic to bypass the frontage road signal and weave with the mainlane traffic. The "X" ramp pattern may cause some drivers to miss an exit located well in advance of the cross street when the exit location is unexpected.

Spread diamond. The spread diamond (Figure 3-24(D)) moves the frontage roads outward to provide better intersection sight distance at the cross street. This design also provides improved operational characteristics with signalized intersections due to the separation between intersections. The additional right-of-way required may limit its usage.

Stacked diamond. Sometimes access to and from the mainlanes is needed on two closely-spaced cross streets. Insufficient distance for consecutive entrance and exit ramps can be resolved by using grade separated or “braided” ramps, resulting in a “stacked diamond” (Figure 3-24 (E)).

Split diamond. In some locations, it may be feasible and desirable to “split” the diamond by having one-way streets for the arterial movement (Figure 3-24 (F)). This is especially true near central business districts where one-way street systems are common. However, the split diamond can also be used to accommodate two closely-spaced two-way arterial roadways crossing a freeway.

Three level diamond. In urban areas, where the cross street carries a high volume of traffic, the three-level diamond interchange, illustrated in Figure 3-25, may be warranted. The through movements of both the controlled access facility and the cross street flow is uninterrupted with only the turning movements requiring regulation by stop signs or traffic lights. This type of interchange is not usually recommended for use as the ultimate design at the crossing of two controlled access facilities because it requires left-turn interchanging traffic to negotiate three traffic signals or stop controls. However, as stage construction for a fully directional interchange between two controlled access facilities, the three-level diamond can be effective. See AASHTO's *A Policy on Geometric Design of Highways and Streets* for design details.

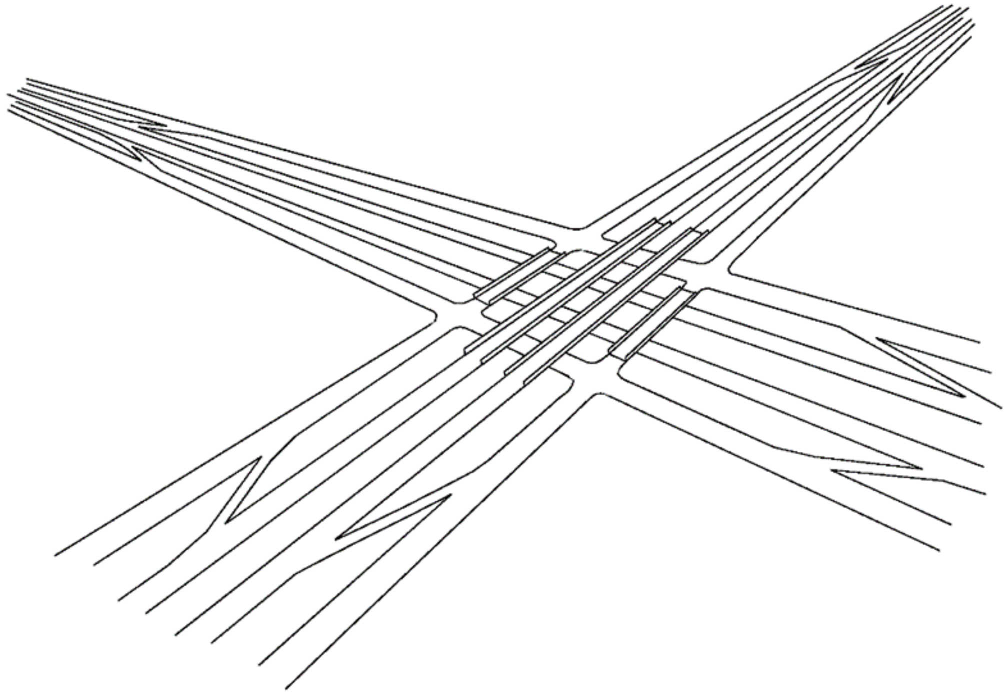


Figure 3-25. Three Level Diamond Interchange.

Single point diamond. A special type of freeway-to-arterial interchange has received attention during recent years and is worthy of discussion. AASHTO's *A Policy on Geometric Design of Highways and Streets* refers to it as a "single point diamond" (SPDI) or "single point urban" (SPUI) interchange. The primary features of an SPDI are that all four turning movements are controlled by a single traffic signal and opposing left turns operate to the left of each other. In this type of interchange, the freeway mainlanes may go either over or under the crossing arterial and the turn movements occur at-grade on the arterial, as illustrated in Figure 3-26. This type of interchange has application only in specialized locations. Traffic operations and signalization must be carefully modeled prior to final design selection of the single point urban interchange.

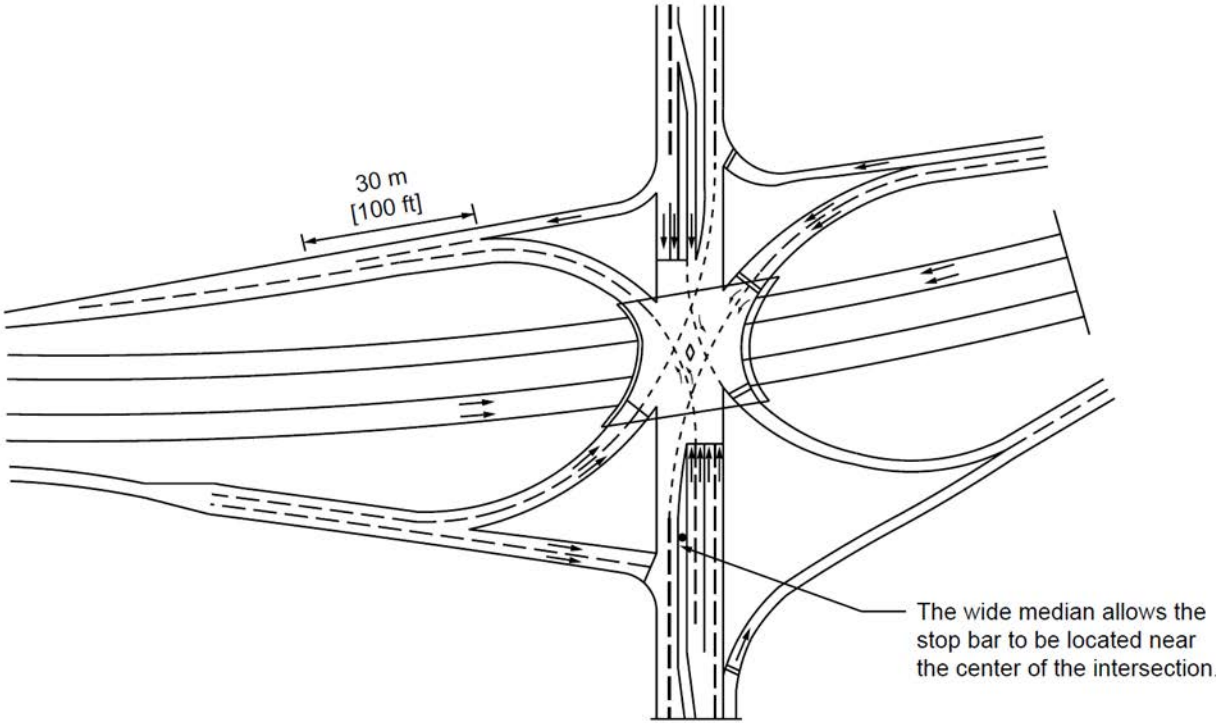


Figure 3-26. Single Point Diamond Interchange.

Three level stacked diamond. The three-level stacked diamond interchange is also an interchange requiring only one signalized intersection. In a sense, it is a three-level version of the “single point diamond” configuration, as illustrated in Figure 3-27. This design grade separates both roadways, and accommodates turning movements with signal operations requiring only one signalized intersection. The two-phase signal operation at the intersection typically provides a level of throughput on the turning movements between a conventional diamond interchange and a fully directional interchange. Furthermore, it works best at separating high arterial cross-street and freeway traffic. It has the same shortcomings as the “single point diamond” in the way it brings the left turn movements together.

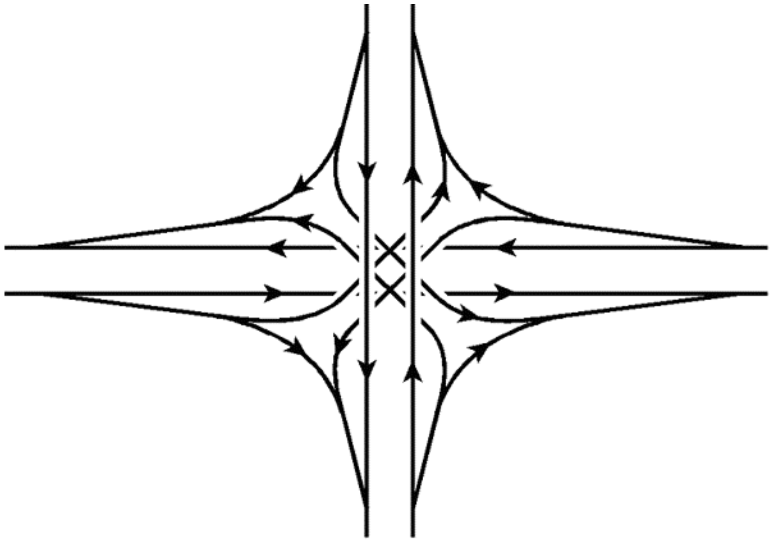


Figure 3-27. Three Level Stacked Diamond Interchange (see Figure 3-28 for At-Grade Portions of the Interchange).

As indicated in Figure 3-28, vehicles enter the intersection with oncoming vehicles to the right in contrast to the left as is the case on conventional diamond interchange intersections. Also, the design is less attractive with continuous frontage roads.

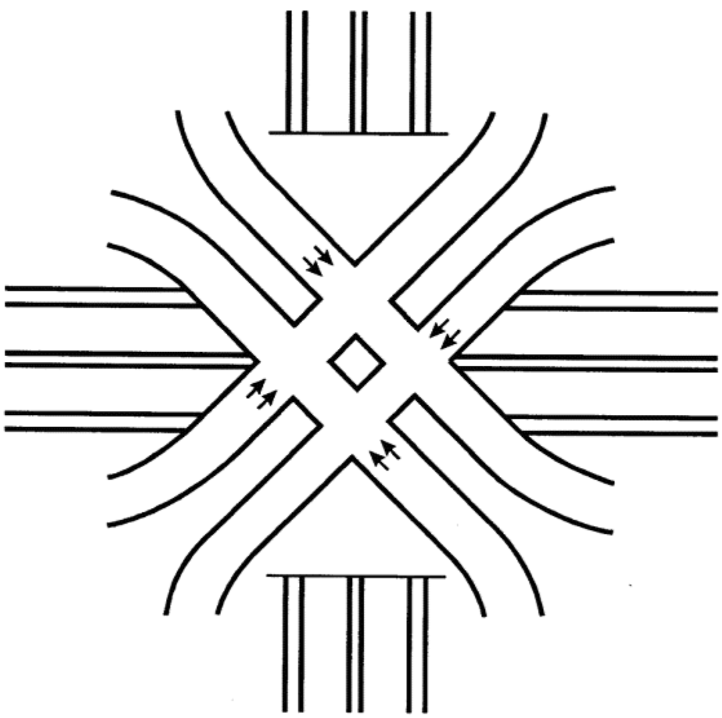


Figure 3-28. Three Level Stacked Diamond At-Grade Interchange.

Cloverleaf Interchanges

Cloverleaf interchanges are very common in many states. These types of interchanges were popular in the early era of freeway construction but are usually no longer considered preferable for freeway to freeway movement, especially when interchange volumes are high. However, in some instances they may be appropriate when a freeway interchanges with a non-controlled access facility in a location away from an urban or urbanizing area. Cloverleafs should not be used where left-turn volumes are high, exceeding 1200 pcph, since loop ramps are limited to one lane of operation and have restricted operating speeds.

Primary disadvantages of the cloverleaf design include the following:

- Large right-of-way requirements;
- Capacity restrictions of loops, especially if truck volumes are significant;
- Short weaving length between loops; and
- Truck difficulty in weaving and accelerating.

When used, cloverleaf designs should include collector-distributor roads to provide more satisfactory operations as further noted in the section on Collector-Distributor Roads.

Full cloverleaf. The four-quadrant, full cloverleaf, illustrated in Figure 3-29, eliminates all left-turn conflicts through construction of a two-level interchange.

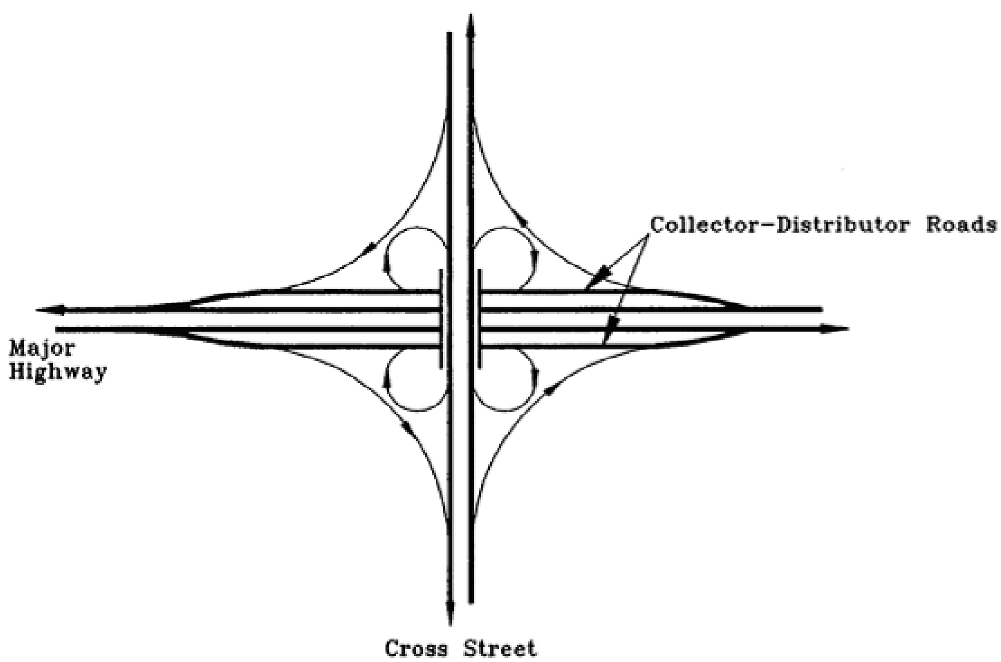


Figure 3-29. Full Cloverleaf Interchange.

Partial cloverleaf. A cloverleaf without ramps in all four quadrants, illustrated in Figure 3-30, is sometimes used when site controls, such as railroads or streams running parallel to the cross street, limit the number of loops. Alternatively, this can be used when the left-turn conflicts caused by the absence of one or more loops are within tolerable limits. With such an arrangement, left-turn conflicts at the ramp intersections require that satisfactory approach sight distance be provided. Several variations on partial cloverleafs are also discussed in the AASHTO's *A Policy on Geometric Design of Highways and Streets*.

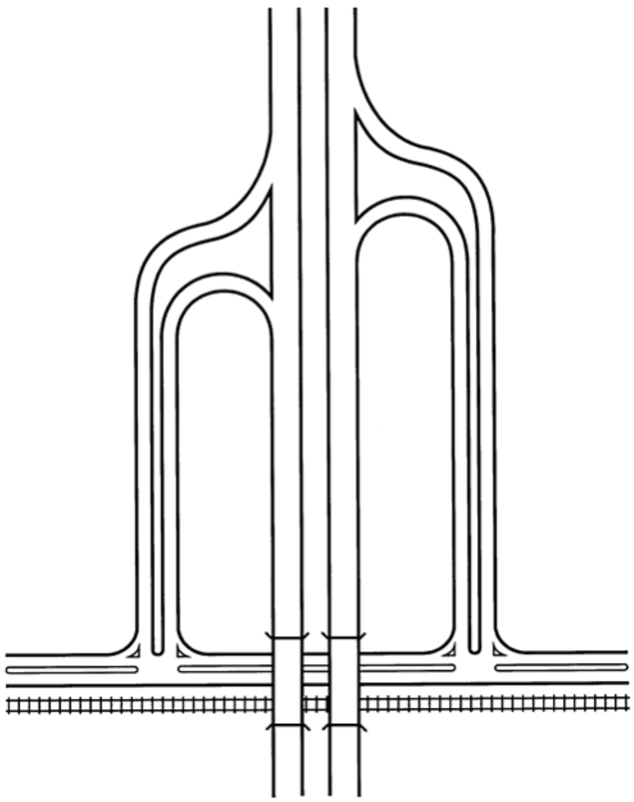


Figure 3-30. Partial Cloverleaf Interchange.

Directional Interchanges.

Interchanges that use direct or semi-direct connections for one or more left-turn movements are called "directional" interchanges (Figure 3-31). When all turning movements travel on direct or semi-direct ramps or direct connections, the interchange is referred to as "fully directional". These connections are used for important turning movements instead of loops to reduce travel distance, increase speed and capacity, reduce weaving and avoid loss of direction in traversing a loop. "Fully directional" interchanges are usually justified at the intersection of two freeways.

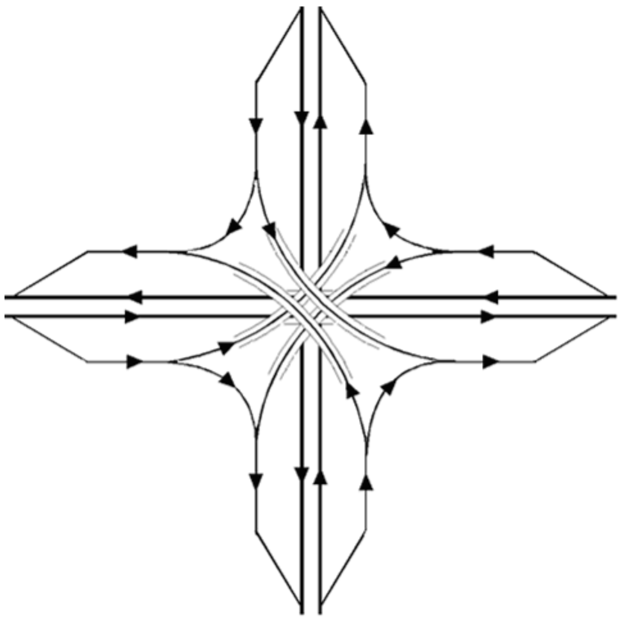


Figure 3-31. Four Level Fully Directional Interchange without Frontage Roads.

Four level without frontage roads. The four-level directional interchange as depicted in Figure 3-31 includes direct connections for all freeway-to-freeway movements, without continuation of any frontage roads through the interchange.

Five level with frontage roads. In some instances, it may be desirable to continue the frontage roads through the interchange at the first or second level, producing a five-level directional interchange. Where frontage roads are made continuous through the interchange, the lower three levels are a three-level diamond configuration. Where stage construction is desired, the three-level diamond will adequately serve moderate traffic volumes until the upper two levels of direct connections are constructed to complete the five-level interchange. Figure 3-32 depicts a five level interchange with frontage roads.

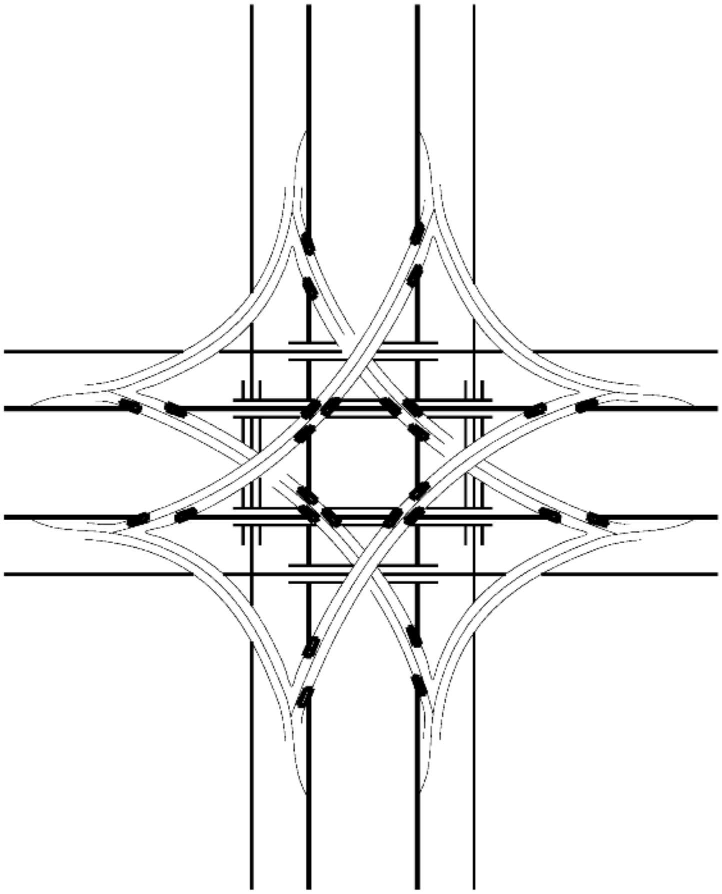


Figure 3-32. Five Level Fully Directional Interchange with Frontage Roads.

Interchange Spacing

Interchange spacing can have a major influence on freeway operations. Appropriate spacing of interchanges is impacted by several items including interchange type, lane configuration, weaving volumes, signing, signal progression, and lengths of speed-change lanes.

The minimum interchange spacing in rural areas is 2 miles. In urban areas, the desirable interchange spacing is 1-mile. Where closer spacing is desired or required, the use of braided ramps or collector-distributor roads is recommended. Braided ramps are ramps that cross over each other and are vertically separated. These ramps separate incoming and exiting traffic by having one ramp pass over the other, thereby eliminating traffic weaving, improving safety and easing congestion.

Traffic operational analysis will be required where lower values of interchange spacing are used.

Interchange spacing is measured between cross streets as shown in Figure 3-33.

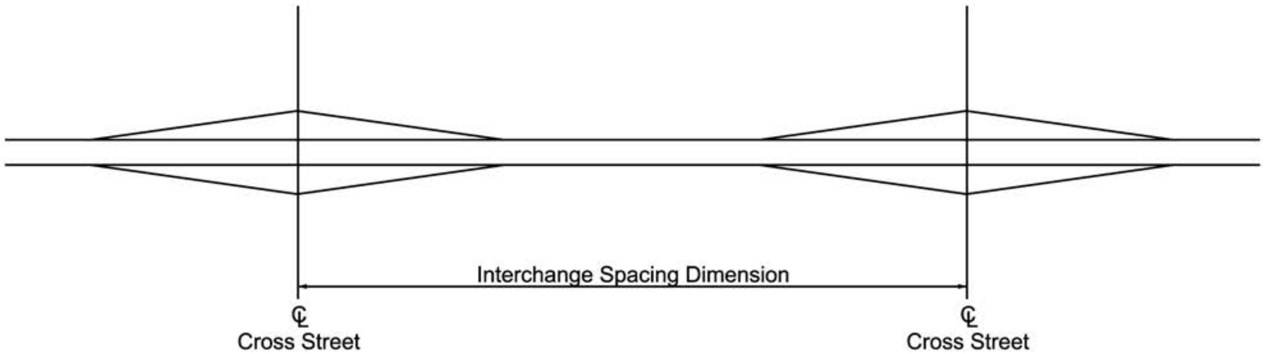


Figure 3-33. Interchange Spacing as Measured between Successive Cross Streets.

Auxiliary Lanes

At interchanges, an auxiliary lane is a full width travel lane that is developed to facilitate traffic operation. Auxiliary lanes are most often used to:

- Accommodate lane balance needs;
- Accommodate slower traffic;
- Accommodate speed changes and weaving;
- Accommodate traffic pattern variations at interchanges;
- Facilitate maneuvering of entering and exiting traffic; and
- Simplify traffic operations by reducing the number of lane changes.

Continuous auxiliary lanes should be constructed between the entrance and exit terminals of interchanges where the distance between the entrance ramp and the exit ramp is short. An auxiliary lane is recommended between the acceleration and deceleration lanes when the distance between successive painted gore noses is less than 2,000-ft.

An auxiliary lane may be introduced as a single exclusive lane or in conjunction with a 2-lane entrance. See AASHTO's *A Policy on Geometric Design of Highways and Streets* for more information concerning the termination of auxiliary lanes. The design of auxiliary lanes should be based on the traffic volumes for the exiting, entering and through movements.

Shoulder and lane widths for auxiliary lanes are shown in Table 3-18.

Ramps and Direct Connectors

This subsection discusses ramps and direct connectors and includes information on the following topics:

- General Information;

- Design Speed;
- Ramp Geometrics;
- Cross Section and Cross Slopes;
- Sight Distance;
- Gores;
- Ramp Terminal Design;
- Ramp Spacing; and
- Metered Ramps.

General Information

All ramps and direct connectors should be designed for one-lane operation with provision for emergency parking; however, if the anticipated volume exceeds the capacity of one freeway lane, two-lane operation may be provided with consideration given to merges and additional entry lanes downstream.

Once ramp locations have been identified on a schematic layout have been exhibited at a public hearing or have otherwise become a matter of public record, extreme caution should be exercised in making any subsequent changes in ramp location to better serve areas that may have developed after the original design was determined. In all cases, proposed changes should be submitted to the Design Division, and another public hearing may be required.

Right-side ramps are markedly superior in their operational characteristics and safety to those that leave or enter on the left side. With right-side ramps, merging and diverging maneuvers are accomplished into or from the slower moving right travel lane. Since a high majority of ramps are right-side, there is an inherent expectancy by drivers that all ramps will be right-side, and violations of driver expectancy may adversely affect operation and safety characteristics.

Direct access to and from ramps can seriously impair safety and traffic operations and, therefore, should not be permitted. Direct access to an interstate interchange connector is prohibited. Direct access to a freeway ramp without a frontage road not access controlled by designation is discouraged and, if allowed, the location should meet TxDOT guidelines and be a safe and sufficient distance from the mainlane ramp gore.

For Interstate ramps, access along the ramp to or from a private drive, business or street is prohibited. The ramp is considered an extension of the mainlanes. 23 CFR 625.3 and 625.4 require adherence to the policies in AASHTO's manual "*A Policy on Design Standards-Interstate System*, May 2016". Specifically, access to the Interstate system, including ramps, shall be fully controlled. Access control shall extend the full length of ramps and ramp terminals at the crossroad or frontage road.

Design Speed

All ramps and connections should be designed to enable vehicles to leave and enter the traveled way of the freeway at 85 percent (desirable) to 70 percent (usual) of the freeway's design speed, rounded up to the nearest 5 mph increment, and limiting the speed differential to 10 mph on the upper range and 20 mph for the mid-range. The upper and mid-range design speeds are not always practical (e.g. constrained right-of-way or retrofit to existing frontage roads, etc.) and lower design speeds may be selected upon review from Design Division, but they should not be less than the lower range presented in Table 3-20. However, every effort should be made to meet the desirable ramp/connector design speed. These speeds do not pertain to ramp terminals, which should be properly transitioned and provided with speed-change facilities adequate for the highway speed involved.

Where the ramp joins a frontage road, the design speed over the length of the ramp may vary, with the portion of the ramp closer to the frontage road being designed to the frontage road speed (i.e. the lower speed). The independent design speed of the ramp should normally be assumed to be from the end of one physical nose to the beginning of the opposite physical nose. The design speed for a ramp should not be less than the design speed on the intersecting frontage roads. AASHTO's *A Policy on Geometric Design of Highways and Streets* provides additional guidance on the application of the ranges of ramp design speed shown in Table 3-20:

Table 3-20: Guide Values for Ramp/Connection Design Speed as Related to Highway Design Speed[1]

Ramp/Connector Design Speed[2] (mph)	Highway Design Speed (mph)										
	30	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**
Upper Range (85%)	25	30	35	40	45	50	50	55	60	65	70
Mid-Range (70%)	20	25	30	30	35	40	45	45	50	55	60
Lower Range (50%)	15	20	20	25	25	30	30	35	35	40	40

Notes:
1. For corresponding minimum radius, see Table 2-3 and Table 2-4.
2. Upper-range values of design speed generally are not attainable on loop ramps. For highway design speeds above 50 mph, loop design speed should be no less than 20 mph.

Ramp Geometrics

Entrance Ramp Connections. The horizontal geometry of entrance ramps will depend on the design speeds of the ramp, the connecting frontage road, and the connecting freeway. The departure from the frontage road should be a smooth transition appropriate for the design speed and speed classification (low speed/high speed) of the frontage road. The pavement elevation and cross slope of the departing ramp should match the pavement elevation and cross slope of the connecting frontage road up to the physical nose. Once the ramp is independent of the connecting frontage road, the ramp horizontal geometry and cross slope should be set to the design speed of the ramp. Where the ramp connects to the freeway, beyond the physical nose, the pavement elevation and cross slope of the entering ramp should match the pavement elevation and cross slope of the connecting freeway.

Exit Ramp Connections. The horizontal geometry of the exit ramps will depend on the design speeds of the ramp, the connecting freeway, and the connecting frontage road. The departure from the freeway may be either tangent or parallel. The pavement elevation and cross slope of the departing ramp should match the pavement elevation and cross slope of the connecting freeway up to the physical nose. Once the ramp is independent of the connecting freeway, the ramp horizontal geometry and cross slope should be set to the design speed of the ramp. Where the ramp connects to the frontage road, beyond the physical nose, the pavement elevation and cross slope of the exiting ramp should match the pavement elevation and cross slope of the connecting frontage road.

Gores

The term "gore" indicates an area at the junction of entrance or exit ramps with the mainlane or frontage roads which is illustrated in Figure 3-34.

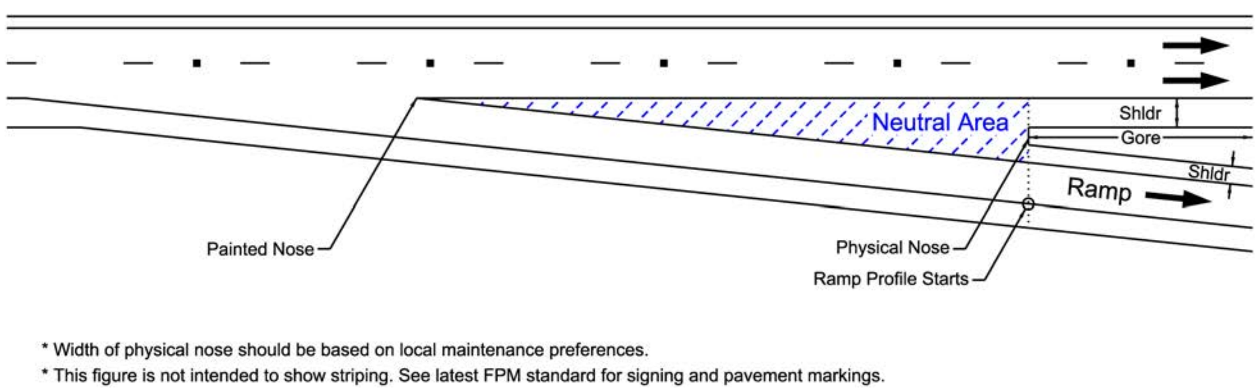


Figure 3-34. Typical Gore Area Characteristics.

For an exit ramp junction at the mainlane, the physical nose is a point downstream from the painted nose of the gore, having some dimensional width that separates the roadways. The painted nose is a point, with no dimensional width, occurring where the travel ways intersect for the mainlane and ramp. The neutral area refers to the triangular area between the painted nose and physical nose.

The geometric layout of these is an important part of exit ramp terminal design. It is the decision point area that should be clearly seen and understood by approaching drivers and it should have a geometric shape appropriate for the likely speeds at that point. In a series of interchanges along a freeway, the gores should be uniform and have the same appearance to drivers.

The gore area, and the unpaved area beyond, should be kept as free of obstructions as practical to provide a clear recovery area. The unpaved area beyond the physical nose should be graded to be as nearly level with the roadways as practical so that vehicles inadvertently entering will not be overturned or abruptly stopped by steep slopes.

Grades and Profiles. Design controls for crest and sag vertical curves on ramps and direct connectors may be obtained from Figure 2-6 and Figure 2-7. Longer vertical curves with increased stopping sight distances should be provided wherever possible. Refer to Chapter 2 for guidance on grade breaks without a vertical curve.

The tangent or controlling grade on ramps and direct connectors should be as flat as possible, and preferably should be limited to 4 percent or less. Certain geometric constraints or topographic conditions permit the use of the following grades in Table 3-21.

Table 3-21: Desirable Maximum Grades for Ramps

Ramp Design Speed (mph)	Max. Grade (%)
25 to 30	7
35 to 40	6
Greater than or equal to 45	5

The ramp should have an independent profile from physical nose to physical nose. The profile grade within the neutral area should be designed as a projection of the connecting road profile and cross slope. Figure 3-35 shows typical characteristics of ramp profile limits.

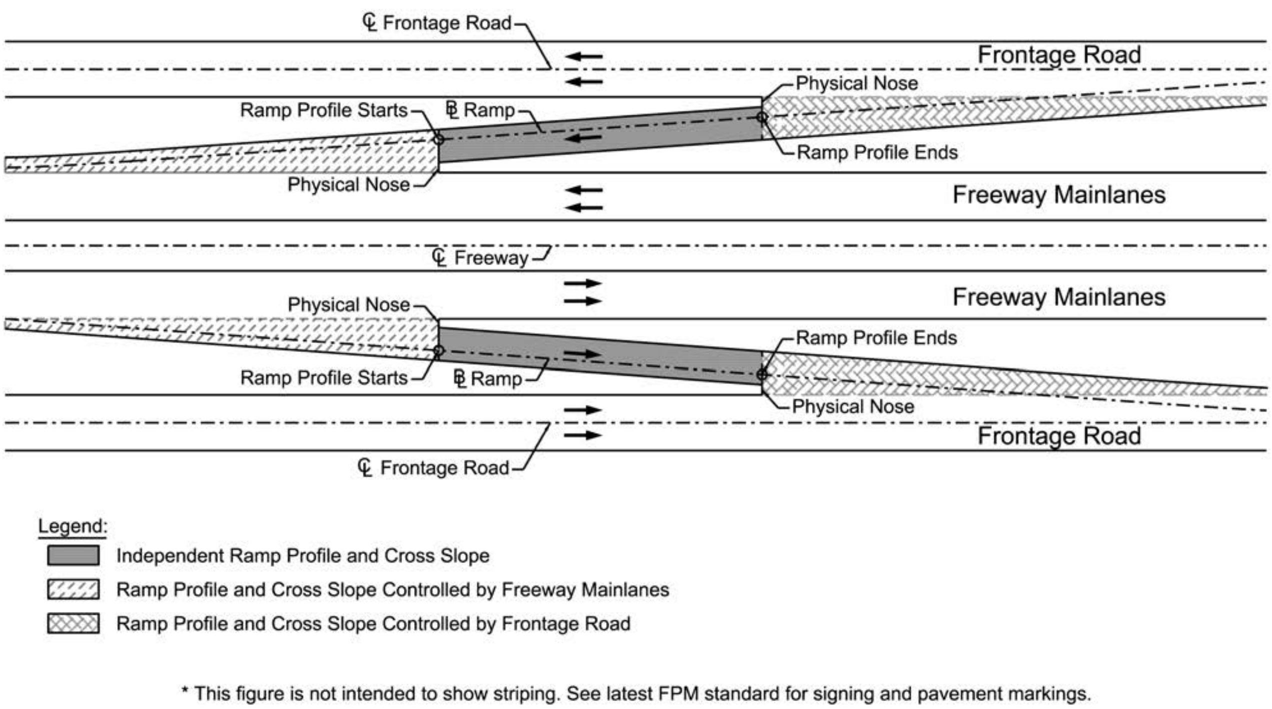


Figure 3-35. Typical Ramp Profile Characteristics.

In some cases, it may be impractical to match the projected roadway profile and cross slope when designing the ramp proper profile. For these situations, a separated median area, with mountable curb or barrier, that warps the neutral area to join the projected profile of the connecting roadway and ramp proper may be considered. If a raised median is not preferred, surfaced mounted delineators may be used within the warped neutral area to discourage drivers from traversing this part of the ramp. See Figure 3-36 for illustrations of a warped neutral area and Table 3-22 for maximum algebraic differences in cross slope.

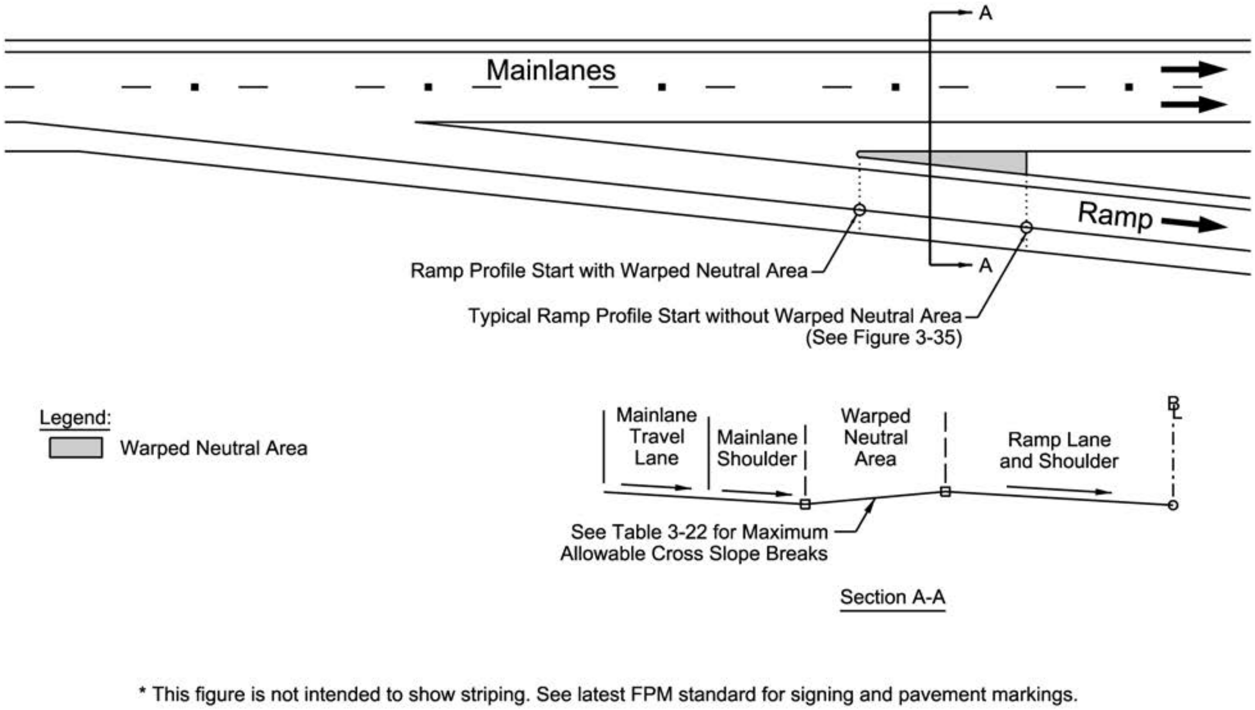


Figure 3-36. Warped Neutral Area.

Table 3-22: Maximum Algebraic Differences in Pavement Cross Slope at Connecting Roadway Terminals

Design Speed of Exit or Entrance Curve (mph)	Maximum Algebraic Difference in Cross Slope at Crossover Line (%)
Less than or equal to 20	5.0 to 8.0
25 to 30	5.0 to 6.0
Greater than or equal to 35	4.0 to 5.0

Cross Section and Cross Slopes

Superelevation rates, as related to curvature and design speed of the ramp or direct connector, should be based on values provided in Table 2-3 or Table 2-4. Table 2-3 may be used for low speed urban facilities and when connecting to frontage roads or cross streets that are also governed by the low speed urban criteria. If the frontage road or cross street are governed by the high-speed non-urban Table 2-4, then the entire ramp should be similarly designed.

The cross section of a ramp or direct connector is a function of the following variables:

- Number of lanes determined by traffic volume;
- Minimum lane and shoulder width, see Table 3-18;
- Lane balance; and
- Where traffic volumes require two lanes merging onto a freeway adequate weaving distance-must be provided.

Lane and Shoulder Widths. Lane and shoulder widths for ramps and direct connectors are shown in Table 3-18.

Sight Distance

Sufficient sight distance should be provided for the probable vehicle speeds on ramps and direct connectors, taking into consideration the grade, vertical curve, alignment, and lateral and corner obstructions to vision. Sight distance and sight lines are especially important at merge points for ramps and mainlanes or between individual ramps. Table 2-1 shows recommended stopping sight distances for ramps and direct connections.

The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum stopping sight distance for the freeway design speed, preferably by 25 percent or more. The Decision Sight Distance, as discussed in Chapter 2, is a desirable goal.

Ramp Terminal Design

The design of ramp terminals for exits from and entrances onto freeways and frontage roads has a significant impact to safety, operation, and capacity of the freeway facility. The following ramp terminal design features are discussed:

- Entrance Ramp Acceleration Lane to Freeway;
- Taper-type Entrance Ramp to Freeway;
- Parallel-Type Entrance Ramp to Freeway;
- Entrance Ramp to Freeway on a Curve;
- Exit Ramp Deceleration Lane from Freeway;
- Taper-Type Exit Ramp from Freeway;
- Parallel-Type Exit Ramp from Freeway;
- Entrance Ramp from Frontage Road;

- Exit Ramps to Frontage Roads;

Entrance Ramp Acceleration Lane to Freeway. The acceleration lane associated with entrance ramps to freeway begins where the driver transitions from the ramp curvature to the flatter geometry of the speed change lane (SCL). The lane allows for acceleration and checking for gaps in the freeway traffic. The length of the acceleration lane is determined on the basis that merging vehicles should enter the through lane at a speed approximately equal to the running speed of the freeway. The acceleration lane ends at the beginning of the taper. The taper section begins where the width of the SCL becomes less than 12-ft and ends at the point where the SCL has fully merged with the freeway through lane. The taper section is not included in the length of the SCL. The factors for the calculation of the length of an acceleration lane are: (a) the speed at which drivers merge with through traffic, (b) the speed at which drivers enter the acceleration lane, and (c) the manner of acceleration.

Table 3-13 provides minimum acceleration lane lengths (La) for entrance terminals, applicable to both taper- and parallel-type entrances; adjustment factors for grade effects are shown in Table 3-14. Point A (shown in Figure 3-37) is the point where La is measured from and controls speed on the ramp. La should not start back on the curvature of the ramp unless the radius of the ramp equals 1,000-ft or more. Before La is extended back onto the ramp past the physical nose of the gore, the actual vertical and horizontal alignments of the ramp should be analyzed to determine if acceleration above the ramp design speed can reasonably begin there.

Gap acceptance length (Lg) is defined as beginning where the traveled ways of the mainlane and ramp roadways are a minimum of 2 lateral feet apart and ending where the right edge of the ramp is 12-ft from the right edge of the through lane of the freeway. The minimum gap acceptance length is 300-ft. The value of La or Lg, whichever produces the greater distance, is suggested for use in the design of the ramp entrance.

The two general forms of entering a freeway are through a taper type and parallel type speed change lanes.

Taper-Type Entrance Ramp to Freeway. A taper-type entrance shown in Figure 3-37 merges into the freeway with a long, uniform taper. A desirable rate of taper is approximately 50:1 to 70:1 (Design Speed:1 or DS:1) between the outer edge of the acceleration lane and the edge of the through traffic lane. The rate of taper is measured against the alignment of the mainlanes.

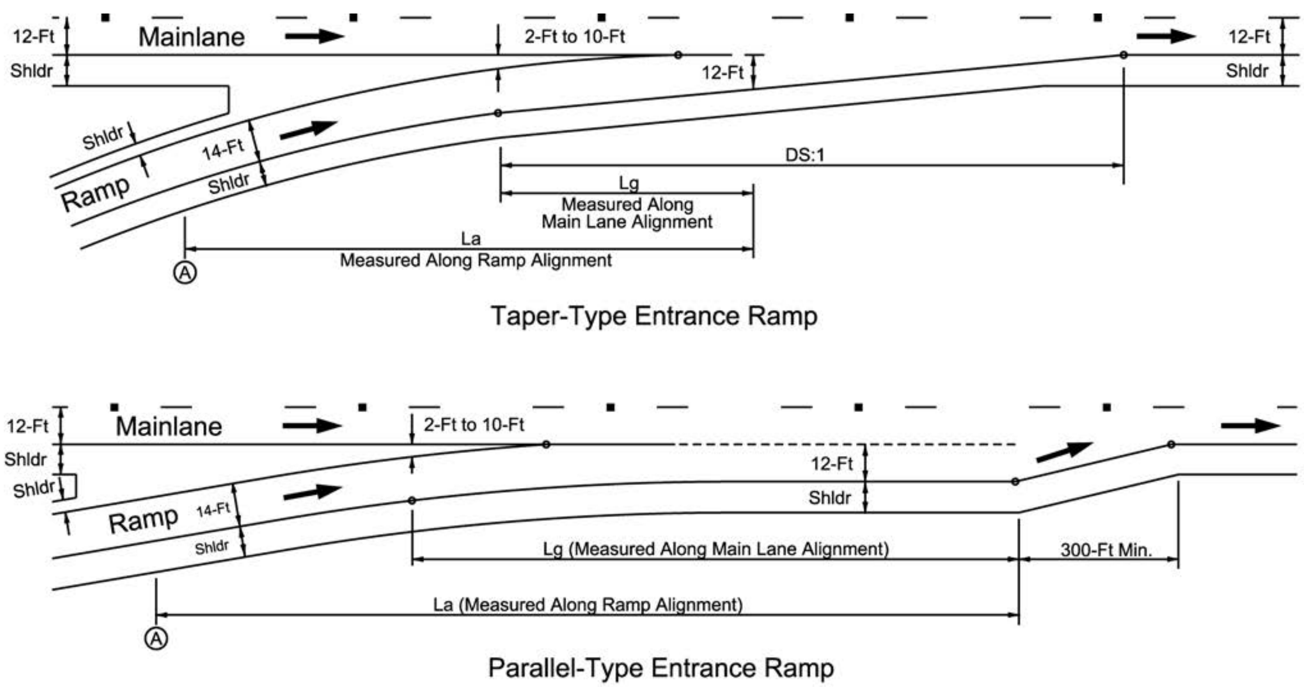
The geometrics should be designed in a way that drivers attain a speed that is within 5 mph of the operating speed of the freeway by the time they reach the point where the left edge of the acceleration lane joins the traveled way of the freeway.

Parallel-Type Entrance Ramp to Freeway. The parallel-type entrance ramp to freeway shown in Figure 3-37 provides an added lane of sufficient length to enable a vehicle to accelerate to near-freeway speed prior to merging. A taper is provided at the end of the added lane.

Desirably, a ramp curve with a baseline radius of 1,000-ft or more and length of at least 200-ft should be provided in advance of the added lane. If a radius less than 1,000-ft is used drivers tend to drive directly onto the freeway without using the acceleration lane. The taper at the downstream end of a parallel-type acceleration lane should be a suitable length to guide the vehicle gradually onto the through lane of the freeway. A minimum taper length of 300-ft is required.

The length of a parallel-type acceleration lane is generally measured from the point where the left edge of the ramp travel way joins the traveled way of the freeway to the beginning of the downstream taper, Lg. Acceleration usually takes place downstream from this point. However, a part of the ramp may also be considered in the acceleration length, La, provided the ramp curve approaching the acceleration lane has a radius of 1,000-ft or more and the motorist on the ramp has an unobstructed view of traffic on the freeway to the motorist's left.

An acceleration length, La, of at least 1,200-ft plus the taper is desirable where the ramp-freeway volume is nearing 4,600 vehicles per hour at the ramp-freeway junction.



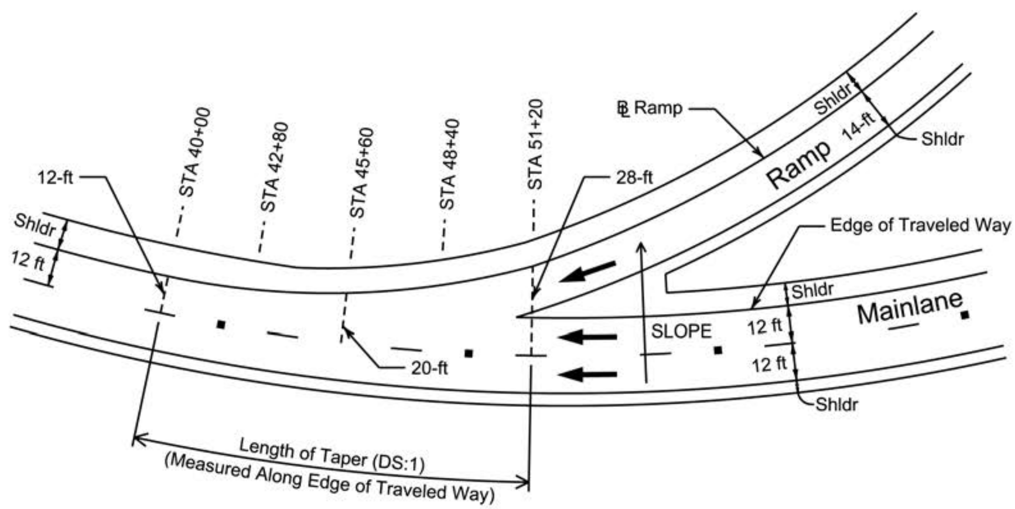
Notes:

1. La = Recommended length of acceleration as shown in Table 3-13 and adjusted in Table 3-14.
2. Point A controls speed on the ramp. La should not start back on the curvature of the ramp unless the radius equals 1,000-ft or more.
3. Lg = Recommended gap acceptance length (300-ft to 500-ft depending on the nose width).
4. The value of La or Lg, whichever produces the greater distance downstream from where the nose equals 2-ft, is suggested for use in the design of the ramp distance.
5. This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 3-37. Single-Lane Entrance Ramp Terminals on a Tangent.

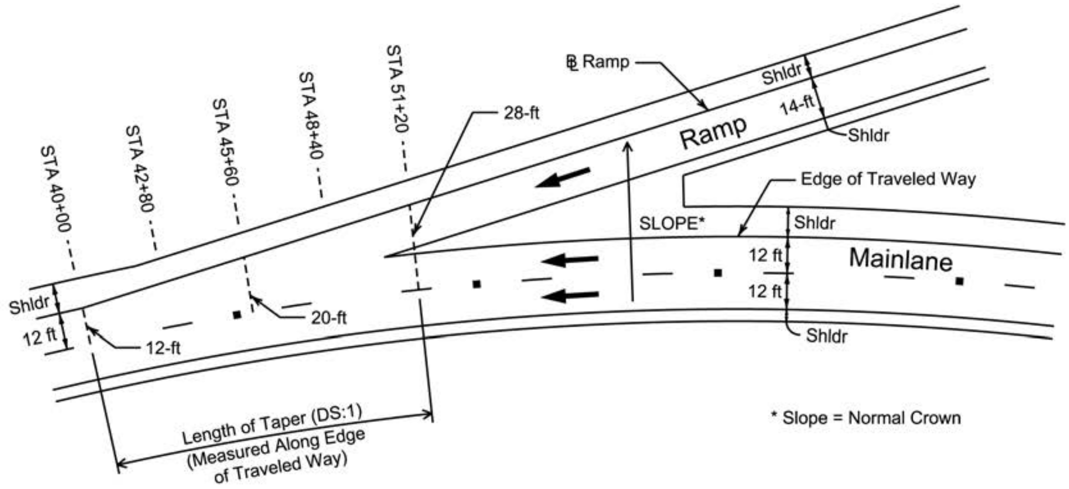
Entrance Ramp to Freeway on a Curve. Connecting taper-type entrance ramps to freeways in a horizontal curve will need to be independently evaluated depending on the radius of the mainlanes as well as any superelevation and/or direction of the cross slope of the mainlanes, if any. See AASHTO's *A Policy on Geometric Design of Highways and Streets* for guidance on developing the alignment of a speed change lane on curves.

Figure 3-38 through Figure 3-40 show different cross slope and curvature combinations of an entrance ramp connecting to the mainlanes. In all scenarios the rate of taper (DS:1) is measured relative to the alignment of the mainlanes. If the mainlanes are superelevated, the cross-slope transition should occur in the ramp prior to the physical nose. In the case where ramp curves are reverse of mainlanes or ramp cross slope exceed mainlane cross slope, sufficient length of tangent or a larger compound curve should be provided on the ramp to accommodate cross slope transitions prior to the physical nose. When a taper-type entrance ramp is used in a curve, the taper should be evenly distributed about the curve to meet the desirable taper length.



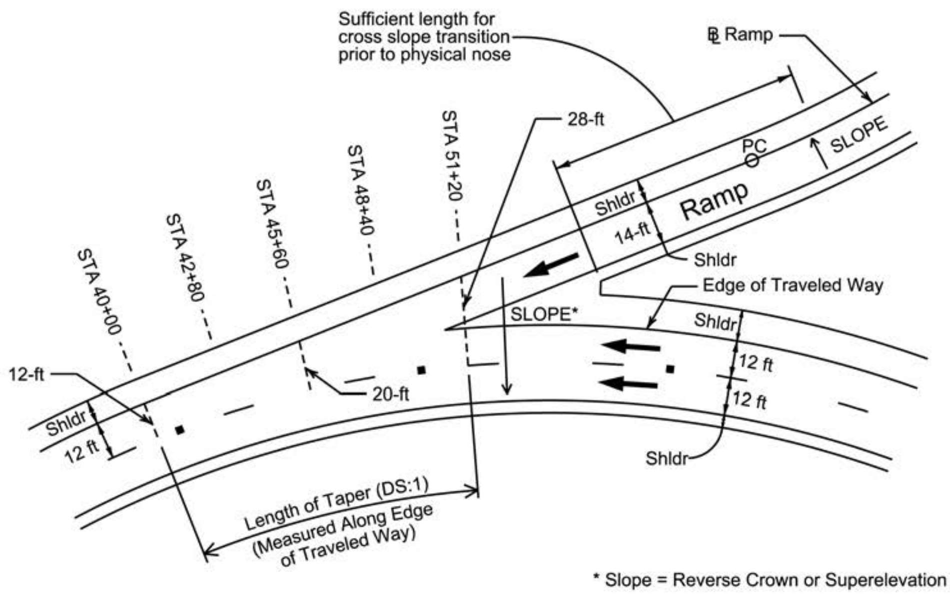
Notes:
1. Length of taper shown is based on a design speed (DS) of 70 mph and a nose width of 2-ft; actual values may vary.
2. This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.
3. The lane widths shown are examples; actual widths may vary.

Figure 3-38. Entrance Ramp to a Freeway Curving In The Direction Of The Ramp.



Notes:
1. Length of taper shown is based on a design speed (DS) of 70 mph and a nose width of 2-ft; actual values may vary.
2. This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.
3. The lane widths shown are examples; actual widths may vary.

Figure 3-39. Entrance Ramp To A Freeway Curving Away From The Direction Of The Ramp.



Notes:
1. Length of taper shown is based on a design speed (DS) of 70 mph and a nose width of 2-ft; actual values may vary.
2. This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.
3. The lane widths shown are examples; actual widths may vary.

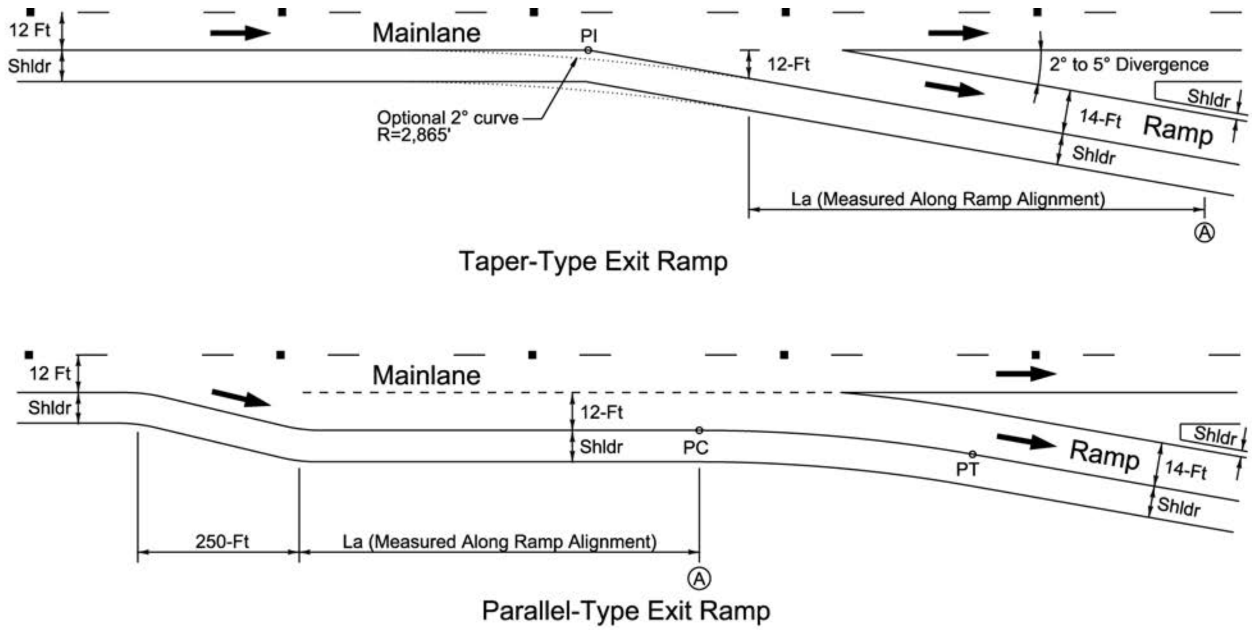
Figure 3-40. Entrance Ramp to a Superelevated Freeway Curving Away from the Ramp.

Exit Ramp Deceleration Lane from Freeway. The deceleration lane associated with exit ramps from freeways is sometimes preceded by a tapered section of roadway. It begins where the speed change lane is 12-ft wide and ends at Point A. The deceleration lane provides a means for vehicles to decelerate from the freeway speed to a speed commensurate with the controlling feature of the ramp. The deceleration lane length (La) is based on three factors: (a) the speed at which drivers maneuver onto the auxiliary lane, (b) the speed at the end of the deceleration lane, and (c) the manner of deceleration.

Taper-Type Exit Ramp from Freeway. A taper-type exit ramp, as shown in Figure 3-41, begins with an abrupt outer edge alignment break providing a clear indication of the point of departure from the through lanes. The divergence angle is usually 2 to 5 degrees. The angular break may be rounded by an appropriate radius at the design engineer's discretion. The divergence angle resulting from the designed alignment should be calculated and compared to the typical 2 to 5 degrees.

Parallel-Type Exit Ramp from Freeway. A parallel-type exit terminal usually begins with a taper, followed by an added lane that is parallel to the traveled way as shown in Figure 3-41. In locations where both the mainlane and ramp carry high volumes of traffic, the deceleration lane provided by the parallel-type exit provides storage for vehicles that would otherwise undesirably queue up on the through lane or on a shoulder. The length of a parallel-type deceleration lane is usually measured from the point where the added lane attains a 12-ft width to the point where the alignment of the ramp roadway departs from the alignment of the freeway. Where the ramp between the gores is curved it is desirable to provide a transition at the end of the deceleration lane.

Minimum deceleration lengths for various combinations of design speeds for the highway and for the ramp roadway are given in Table 3-23. Grade adjustments are given in Table 3-14. The values in Table 3-23 for minimum deceleration lane length on exit ramps do not account for any deceleration in the through lanes, therefore providing a conservative estimate for design. The designer should assume that all deceleration takes place in the speed-change lane when determining the minimum deceleration lane length.



Notes:

1. La = as shown in Table 3-23 with adjustments in Table 3-14.
2. For taper-type exit ramps, if divergence angle is greater than 5°, a designed alignment that meets or exceeds the ramp design speed must be used.
3. Point A is the controlling geometric feature that governs the design speed of the ramp.
4. For parallel-type exit ramps, La may extend beyond the ramp curve if the ramp curve meets or exceeds the design speed of the ramp.
5. This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 3-41. Typical Single-Lane Exit Ramp Terminals.

Table 3-23: Minimum Deceleration Lane Lengths for Exit Ramps with Flat Grades of Less Than 3 Percent

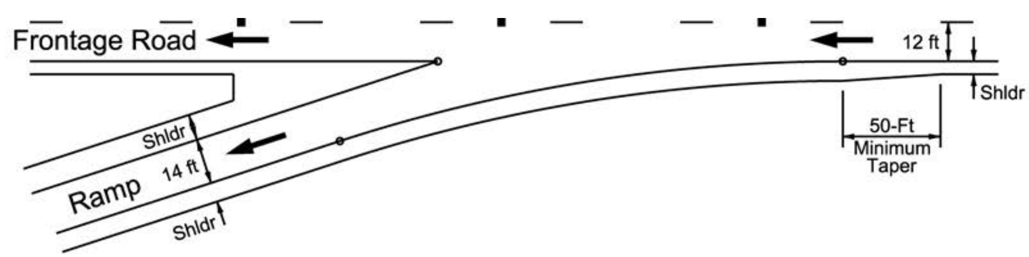
Deceleration Lane Length, La (ft) for Design Speed of Controlling Feature on Ramp, V' (mph)									
Highway Design Speed, V (mph)	**Stop Condition**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
30	235	200	170	140	-	-	-	-	-
35	280	250	210	185	150	-	-	-	-
40	320	295	265	235	185	155	-	-	-
45	385	350	325	295	250	220	-	-	-
50	435	405	385	355	315	285	225	175	-
55	480	455	440	410	380	350	285	235	-
60	530	500	480	460	430	405	350	300	240

Table 3-23: Minimum Deceleration Lane Lengths for Exit Ramps with Flat Grades of Less Than 3 Percent

Deceleration Lane Length, La (ft) for Design Speed of Controlling Feature on Ramp, V' (mph)									
Highway Design Speed, V (mph)	**Stop Condition**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
65	570	540	520	500	470	440	390	340	280
70	615	590	570	550	520	490	440	390	340
75	660	635	620	600	575	535	490	440	390
80	705	680	665	645	620	580	535	490	440

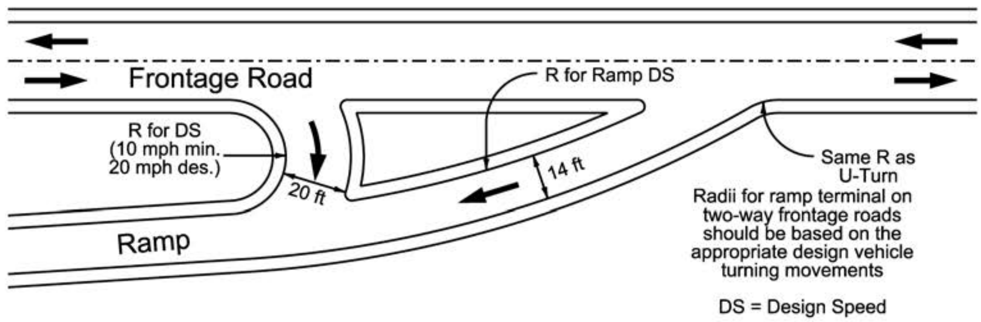
Entrance Ramp from Frontage Road. The geometry of ramps connecting from frontage roads will differ significantly depending on whether they are connecting to a one-way or two-way frontage road. Connection from one-way frontage roads is recommended, as this reduces the chance of wrong-way traffic on a ramp. The horizontal geometry of the ramp connecting from a frontage road should be compatible with the horizontal geometry of the frontage road. If the frontage road is designed based on low-speed urban criteria, then the ramp connection should be similarly designed. If the frontage road is designed on high-speed rural criteria, then the ramp connection should be similarly designed.

While treatment of pavement edges may differ depending on whether the frontage road is shouldered or curbed, the path of the traffic remains the same. The traffic path should follow a smooth horizontal curve tangent to both the inside yellow stripe of the frontage road through to the inside yellow stripe of the ramp. This applies whether or not the traffic is entering from the through lanes on the frontage road or a dedicated auxiliary lane. Geometry guidance for entrance ramps from frontage roads is shown in Figure 3-42 and Figure 3-43. For more geometry and access control information on entrance ramps from frontage roads, refer to Figure 3-18.



Note:
This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 3-42. Taper-Type Entrance Ramp from a One-Way Frontage Road.



Note:
This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 3-43. Taper-Type Entrance Ramp from a Two-Way Frontage Road.

Exit Ramps to Frontage Roads

Ramps should connect to the frontage road at the minimum distance specified in Table 3-16. Greater distances are desirable to provide adequate weaving length, space added for vehicle storage and turn lanes at the cross street. For more geometry and access control information on exit ramps to frontage roads, refer to Figure 3-17.

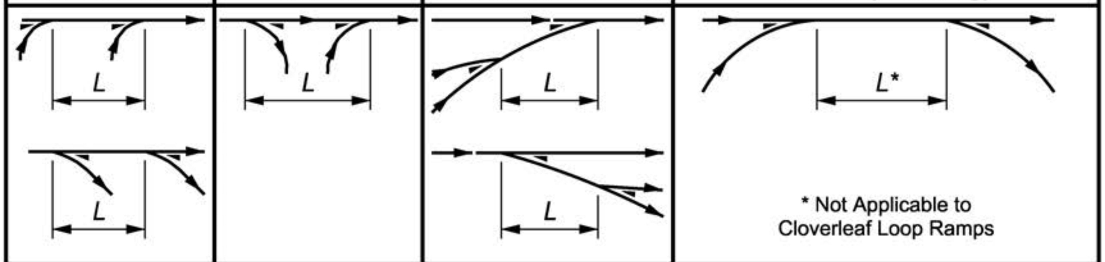
Ramp Spacing

Concerning ramp spacing, the term 'ramp' includes all types, arrangements, and sizes of turning roadways that connect two or more legs at an interchange, including direct connectors and freeway to freeway connections.

The minimum acceptable distance between ramps is dependent upon the merge, diverge and weaving operations that take place between ramps, as well as distances required for signing. The minimum distance is measured from painted nose of gore to painted nose of gore. The distance between successive ramps must be determined with the analysis procedures outlined in the Highway Capacity Manual (HCM). During the HCM analysis it may be determined that an auxiliary lane is needed to meet the appropriate LOS. See Level of Service to determine the appropriate LOS for a facility.

The minimum spacing values shown in Figure 3-44 represent a reasonable starting point during planning and early design. Ramp configurations shown in Figure 3-44 are relative to the facility being traveled on and assume no auxiliary lane is provided. When the distance between the successive painted noses of an entrance-exit (EN-EX) ramp combination is less than 2,000-ft on a freeway or 1,500-ft on a collector-distributor or frontage road, the speed-change lanes should be connected to provide an auxiliary lane. When the minimum distance cannot be achieved, consideration should be given to consolidating or braiding the ramps in lieu of requesting a design waiver.

A design waiver will be required if the minimum lengths specified in Figure 3-44 (without an auxiliary lane) or sufficient LOS is not met.



EN-EN or EX-EX		EX-EN		Direct Connectors		EN-EX (Weaving)			
Freeway	CFR	Freeway	CFR	System Interchange	Service Interchange	A	B	C	D
Minimum Lengths Measured between Successive Ramp Terminals									
1,000-ft	800-ft	500-ft	400-ft	800-ft	600-ft	2,000-ft	1,600-ft	1,600-ft	1,000-ft
Desirable Lengths Measured between Successive Ramp Terminals									
1,500-ft	1,200-ft	750-ft	600-ft	1,200-ft	1,000-ft	3,000-ft	2,000-ft	2,000-ft	1,500-ft

CFR - Collector Distributor or Frontage Road
EN - Entrance
EX - Exit
L - Distance from painted nose to painted nose
System Interchange - Freeway to Freeway Interchange
Service Interchange - Freeway to Non-Freeway Interchange

A = Weaving on a freeway between two System Interchanges or a System Interchange and Service Interchange.
Example 1: A direct connection from freeway 1 followed by a direct connection to freeway 3, while traveling on freeway 2.
Example 2: A direct connection from freeway 1 followed by an exit ramp to a CFR or cross street, while traveling on freeway 2.

B = Weaving on CFR between two System Interchanges or a System Interchange and Service Interchange.
Example 1: A direct connection from freeway 1 followed by an entrance ramp to freeway 2, while traveling on CFR 1.
Example 2: A direct connection from freeway 1 followed by an exit ramp to CFR 2 or a cross street, while traveling on CFR 1.

C = Weaving on a freeway between two Service Interchanges.
Example: An entrance ramp to a freeway followed by an exit ramp from a freeway, while traveling on a freeway.

D = Weaving on a CFR between two Service Interchanges.
Example: An exit ramp from a freeway followed by an entrance ramp to a freeway, while traveling on a CFR.

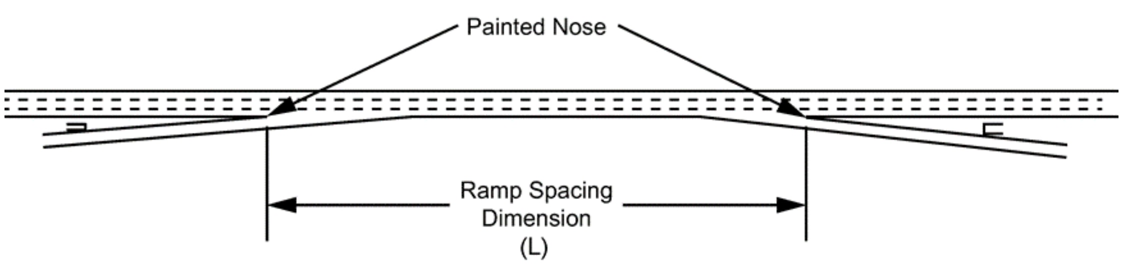


Figure 3-44. Arrangements for Successive Ramps.

Metered Ramps

Where ramps are currently expected to accommodate metering, or expected to in the future, the geometric design features shown in AASHTO’s *Design Criteria for Ramp Metering* should be considered. Ramp metering, when properly designed and installed, has been shown to have benefits for the operation of the mainlanes. However, since ramp meters are installed to control the number of vehicles that are allowed to enter the mainlanes, an analysis of the entire roadway network area should be done to determine adverse operational impacts to other roadways. It is suggested that the analysis specifically include both frontage road and adjacent cross street operations of through traffic, turning movements, and queue lengths. See AASHTO's *A Policy on Geometric Design of Highways and Streets* for additional discussion and design details on ramp metering.

Collector-Distributor Roads

A collector-distributor road is a road that parallels and connects the main travel lanes and frontage roads or entrances ramps. This is sometimes referred to as an "Intersection Bypass". A collector-distributor is similar to an exit ramp but is typically longer than a ramp. A collector-distributor road is used to meet the following goals:

- Minimize weaving along the mainlanes and/or frontage roads;
- Optimize capacity and improve traffic operations at interchanges;
- Enhance safety by managing traffic speeds of the connected facilities;
- Minimize number of the mainlane ramps; and
- Provide exit from the main lanes in advance of cross roads.

Collector-distributor roads are a component at all cloverleaf interchanges. Where there is considerable demand for frequent ingress and egress, as in and near the business districts of large cities, a collector-distributor road, continuous through several interchanges, should be provided.

Collector-Distributor Design Criteria:

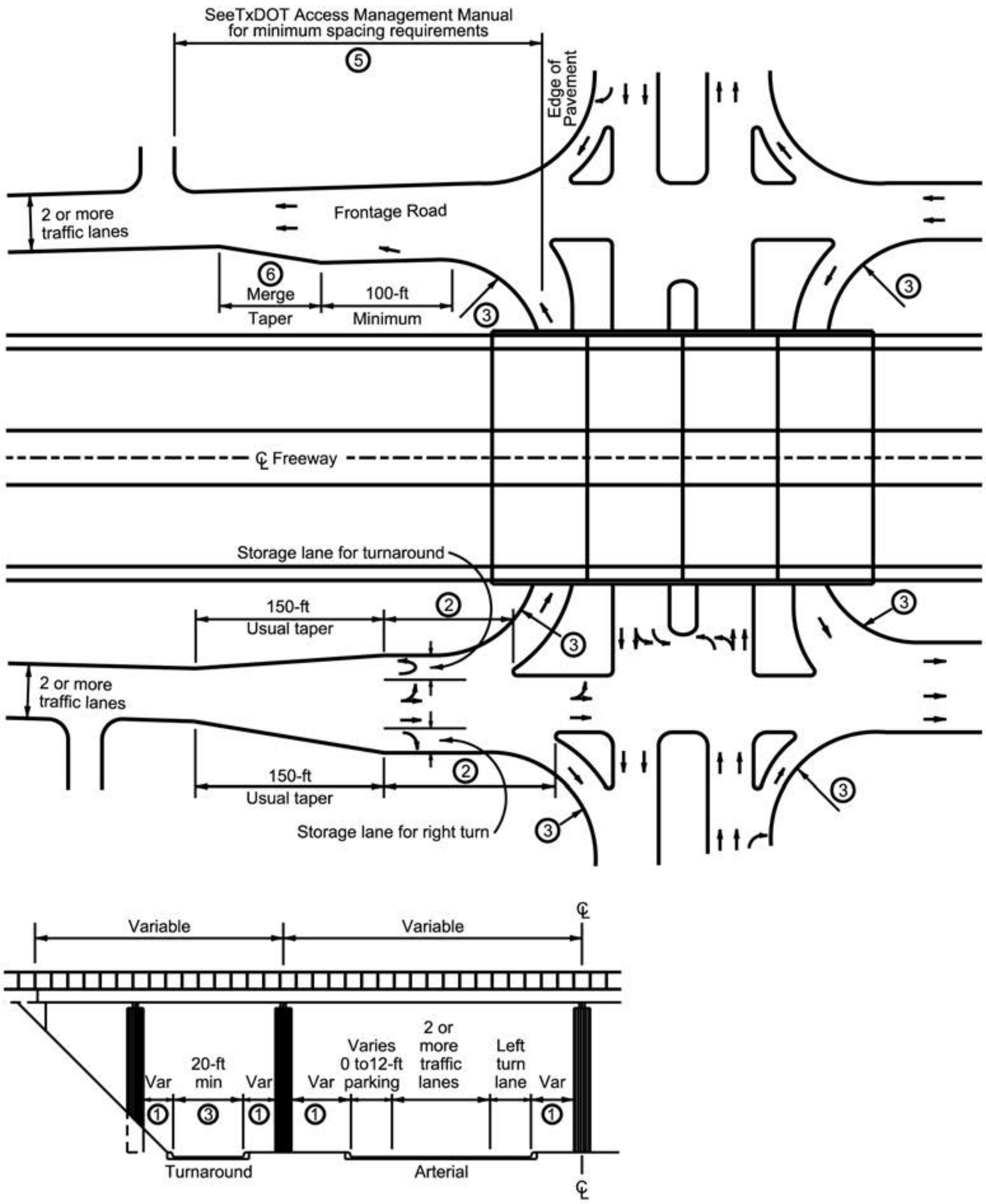
- Design Speed: Table 3-20 (ramp criteria);
- Roadway Width: Table 3-18 (ramp criteria); and
- Capacity analysis and basic lane determination should be performed for the overall system rather than for the separate roadways.

Frontage Road Turnarounds and Intersection Approaches

Turnaround lanes are to be provided at all interchanges with major arterials in urban and suburban areas where the freeway lanes are flanked by one-way frontage roads. Turnaround lanes are not to be provided where two-way frontage roads are used. In urban and suburban areas, overpasses should be arranged so that turnarounds may be added in the future. This includes provisions for end spans and vertical clearance for future turnarounds at overpasses. Underpasses should also allow for vertical clearances on future elevated turnarounds.

When the cross street overpasses the freeway, the resulting turnarounds will be on bridge structures. In these cases, sight lines and distances should be carefully evaluated with respect to any bridge railing sight obstructions. Similarly, sight lines and distances for cross street underpasses must also be evaluated for bridge header and retaining wall obstructions.

Figure 3-45 shows a typical example of a diamond interchange with frontage roads and turnarounds.



① If practical, the face of new bridge columns should be located 6-ft or more from the face of curb. Where potential future expansion of the cross road exists, applicable new bridge columns should be located 16-ft from the curb face.

② Turnaround approach bay length should be 525-ft minimum. For interchanges that exceed a total volume of 40,000 ADT or operate at LOS E or F, a traffic analysis should be performed to determine the required storage bay length.

③ Width and turning radius should be based on appropriate design vehicle.

④ See Figure 3-20 for lane layout option if dual left turn lanes are needed.

⑤ If closing adjacent driveways is not feasible, consider the use of traffic control devices and/or channelization (e.g., pavement markings, flexible pylons, and/or raised curbs) to improve turnaround lane departure operations.

⑥ Refer to TMUTCD for merge taper length. Frontage road design speed should be used to determine length of taper.

Figure 3-45. Typical Diamond Interchange with Frontage Road.

Results from field-based observations, numerous simulations of site improvements with the potential to improve U-turn operations, and a full safety investigation of factors contributing to crashes at interchanges have produced the following guidance on the planning, design, and operation of turnaround lanes: Reference TTI Study No. 06894.

- Turnaround lanes should be considered for future interchanges with a projected (20-year) peak-hour volume of at least 2,000 vph, or roughly 20,000 ADT. For existing interchanges, turnaround lane implementation should be considered when total interchange traffic volume reaches 4,000 vph, or approximately 40,000 ADT;
- Turnaround lane design should include an approach bay with a minimum length of 525 ft. In rural areas, this length primarily provides stopping sight distance on the U-turn approach for higher-speed operations. In urban areas, the bay length requirement is designed to allow U-turning vehicles to avoid interference from left-turn queues in the adjacent lane;
- Operations are improved if the turnaround lane departure features either a full added lane or an acceleration lane (minimum 100-ft length) with taper. Turnaround lane departures featuring stop or yield control, or those that terminate with only a taper transition into a frontage road lane, should only be used where geometric constraints or low-volume conditions exist;
- Unless trucks are specifically prohibited turnaround design should provide adequate turn radii to accommodate heavy vehicles;
- To minimize delay and queuing in the turnaround lane and to minimize the potential for crashes on the frontage road, consider closing driveways within the distances specified in the TxDOT *Access Management Manual*;
- If closing adjacent driveways is not feasible, consider the use of traffic control devices and/or channelization (e.g., pavement markings, flexible delineators, and/or raised curbs) to improve turnaround lane departure operations. Field observation and simulation studies have verified the benefits (e.g., reduced delay and fewer crashes) of constraining weaving maneuvers from turnaround lanes to adjacent downstream driveways;
- Consider right-turn accommodations at the interchange and their impacts on operations and safety. Safety improvements have been observed when right turning traffic yields at the intersection through frontage road traffic (i.e., no right-turn bays or right-turn acceleration lanes were present);
- Signal timing can be used as an interchange management tool to support U-turn operation. Both cycle length and split adjustments have been successfully demonstrated to reduce frontage road queue length and average delay on frontage road approaches. Shorter queue lengths reduce the likelihood of a left-turn queue blocking access to a turnaround lane; and
- Consider the use of dotted line markings to improve interchange operations. Dotted lines to extend lane lines into the intersection and guide drivers through the appropriate turning path have shown reduced delay for turnaround lane movements under medium- to high-volume interchange operations. Directing cross street left-turn vehicles to the middle and/or right frontage road lanes provides gaps in the frontage road for vehicles using the turnaround lane.

Section 7 — Freeway Corridor Enhancements

Overview

This section discusses other transportation modes and includes discussions on where to find information on planning and design criteria for these modes.

Freeways with High Occupancy Vehicle Treatments

High Occupancy Vehicles (HOV) lanes are a commonly used approach in urban freeway environments to reduce congestion and travel times.

Guidelines for the planning and designs of HOV facilities are given in AASHTO's *Guide for the Design of High Occupancy Vehicle Facilities* and in the *Guidance for Future Design of Freeways with High Occupancy Vehicle (HOV) Lanes Based on an Analysis of Crash Data from Dallas, Texas*, by the Texas Transportation Institute (TTI), 2004. Note that a Design Exception would be required if the desirable lane and shoulder widths shown in the AASHTO Guide for the Design of HOV Facilities are not met.

Light Rail Transit

Light Rail Transit systems are being considered in some urban environments as an approach to reducing congestion and travel times.

Guidelines for the incorporation of light rail transit systems in the transportation network are given in the Transit Cooperation Research Report by Lawrence G. Lovejoy, P.E., *TCRP Report 155, Track Design Handbook for Light Rail Transit, Second Edition*, Transportation Research Board, National Academy Press, Washington, D.C., 2011.

Peak Hour Lanes

Part-time shoulder use is a transportation system management and operation (TSM&O) strategy that allows use of the left or right shoulders as travel lanes during some, but not all, hours of the day.

Guidelines for the use of part time shoulders are given in *Use of Freeway Shoulders for Travel - Guide for Planning, Evaluating, and Designing Part-Time Shoulder Use as a Traffic Management Strategy*, FHWA-HOP-15-023, Federal Highway Administration, Washington, D.C., 2016.

Tolled Express Lanes

Tolled express lanes are typically a “freeway-within-a-freeway” where the express lane(s) are separated from the general purpose lanes. A number of design approaches are available depending on policies for pricing, vehicle eligibility, and access control.

Guidelines for the implantation of tolled express lanes can be found in *Managed Lanes Primer*, FHWA-HOP-05-031, Federal Highway Administration, Washington, D.C., 2008.

Section 8 — Texas Highway Freight Network (THFN)

Overview

On September 28, 2017 a memorandum was distributed by Bill Hale, P.E. (Chief Engineer) that provided general guidance for the TxDOT Freight Vertical Clearance Policy. The purpose of the Freight Policy is to make Texas the leader in providing movement of freight in the nation. This is a process that should be approached with a long-range vision and plan in place to provide a continuum of efficient Freight movement within the State. Additional consideration and planning should also be given to existing bridges that act as Freight bottlenecks due to low vertical clearances, even if there are no immediate plans for new construction or reconstruction of these bridges. Consideration for exchanging overpasses for underpasses should be given as well.

This policy is applicable to projects that meet the following criteria:

- Let on September 1, 2020 or later;
- Designated as being on the THFN as shown on the THFN map maintained by the Transportation Planning and Programming Division (TPP); and
- Bridge new construction or reconstruction; including bridge widening. BMIP and maintenance projects may not be considered bridge reconstruction, but this would need to be handled on a case-by-case basis in consultation with the Bridge Division. Redecking a bridge will also be handled on a case-by-case basis in consultation with the Bridge Division.

Note, the policy does not apply to THFN overpasses, frontage roads, direct connectors off of the THFN, and entrance and exit ramps that include bridge underpasses. The THFN vertical clearance requirements should be considered for bridge and incidental vertical obstruction to the Freight Network that support significant origin/destination locations.

Vertical Clearance at Structures

Vertical clearance requirements for roadway structures can be found in Table 2-11.

Minimum vertical clearances for bicycle and pedestrian crossover structures is are increased 1.0-ft from the minimum vertical clearance requirement for vehicular structures due to the increased risk of personal injury upon impact by over-height loads and the relative weakness of such structures to resist lateral loads from vehicular impact.

Signs, Overhead Sign Bridges (OSB’s), Signals

For the designated THFN, overhead signs shall provide a vertical clearance of not less than the requirement in Table 2-11 to the sign, light fixture, or sign bridge over the entire width of the pavement and shoulders. For traffic signals on the THFN, the bottom of the signal housing and any related attachments to a vehicular signal face located over any portion of a highway that can be used by motor vehicles shall not be less than 0.5-ft below the requirements in Table 2-11. See the latest version of the TMUTCD and applicable TxDOT standards for additional guidance.

Other Overhead Utilities

All overhead utilities over the designated THFN project must meet the requirements specified in TxDOT's *ROW Utility Manual*, Chapter 3.

Chapter 4 — Non-Freeway Rehabilitation (3R) Design Criteria

Contents:

Section 1 — Purpose

Overview

Rehabilitation (3R) projects consist of non-freeway transportation projects that extend the service life and enhance the safety of a roadway. In addition to resurfacing and restoration, the activities may include upgrading the geometric design and safety of the facility. Work on 3R projects does not include the addition of through travel lanes (i.e. no added capacity). 3R projects may include upgrading geometric features such as roadway widening, minor horizontal realignment, and bridge improvements to meet current standards for structural loading and accommodate the approach roadway width. See alignment discussion in Chapter 4 Section 2, Design Characteristics for additional clarification on horizontal and vertical alignment.

Design Guidelines

Design guidelines for 3R projects have been developed to allow greater design flexibility. At the District's option, design values above those presented in this chapter may be used.

These guidelines offer sufficient flexibility to ensure cost effective design and further compliance with the program goals of preserving and extending the service life and enhancing safety. While highway safety may not be the primary reason for initiating a 3R project, it is an essential element of all projects. 3R projects should identify and incorporate appropriate safety enhancements.

For 3R projects, current average daily traffic (ADT) volumes of less than 1,500 are defined as low traffic volume roadways.

3R Projects must be assessed to determine if bicycle accommodations are required per Chapter 6, Section 4 (Bicycle Facilities); if bicycle facilities are provided, they must meet the additional requirements specified in Chapter 6, Section 4.

Section 2 — Design Characteristics

Pavement Design

Pavement rehabilitation includes all pavement-related work undertaken to extend the service life of an existing facility. This includes placement of additional surfacing material and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural and/or functional adequacy. The following are some examples of pavement rehabilitation work:

- Resurfacing to provide improved structural capacity and/or serviceability;
- Removing and replacing deteriorated materials;
- Replacing or restoring malfunctioning joints;
- Reworking or strengthening of bases and subbases;
- Recycling existing materials; and
- Adding underdrains.

The existing pavement condition and deficiencies should be identified for 3R projects. Design strategies selected to correct deficiencies will vary from seal coats to overlays to complete pavement structure reconstruction. Projects that consist only of seal coats or overlays, and do not meet the design guidelines presented in this chapter, are not eligible for rehabilitation funding.

Reference TxDOT's *Pavement Manual* for additional information related to pavement rehabilitation.

Geometric Design

Geometric design guidelines are provided for the following roadways in the tables indicated.

- Rural multilane highways, Table 4-1;
- Rural two-lane highways, Table 4-2;
- Urban streets, Table 4-3;
- Rural frontage roads, Table 4-4; and
- Urban frontage roads, Table 4-5.

To measure bridge width on bridges without curbs, measure to the nominal face of rail. Reference TxDOT's Bridge Railing Manual and Bridge Railing Standards for the nominal widths of specific rail types and additional guidance. To measure bridge width on bridges with curbs, measure to the face of curb.

Table 4-1: 3R Minimum Design Guidelines for Rural Multilane Highways (Nonfreeway)[1]

Design Element	Guideline for Highway Class		
	6-Lane Divided	4-Lane Divided	4-Lane Undivided
Design Speed (mph)[2]	50	50	50
Lane Width (ft)	11	11	11
Outside Shoulder Width (ft)	4	4	4
Inside Shoulder Width (ft)	4	2	N/A
Turn Lane Width (ft)[3]	10	10	10
Clear Zone (ft)[4]	16	16	16
Bridges: Width to be retained (ft)[5]	42	28	52

Notes:
1. These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area or to provide operational improvements at specific locations.
2. Considerations in selecting design speeds for the project should include the roadway alignment characteristics as discussed in this chapter. Projects should be designed to target, as close as possible, the existing or anticipated posted speed as practical.
3. For two-way left turn lanes, 11-ft to 14-ft usual.
4. A clear zone is the unobstructed, traversable area provided beyond the edge of the through traveled way for the recovery of errant vehicles. Clear Zone is measured from the edge of travel lane to the obstruction. For low-speed rural collectors, and all rural local roads, a clear zone of 10-ft is allowable in constrained circumstances.
5. Where structures are to be modified, bridges should meet approach roadway width as a minimum. (Approach roadway width is the total width of the lanes and shoulders.) Greater bridge widths may be appropriate if the rehabilitation project increases roadway life significantly or if higher design values are selected for the remainder of the project. Existing structure widths less than those shown may be retained if the total lane width is not reduced across or near the structure.

Table 4-2: 3R Minimum Design Guidelines for Rural Two-Lane Highways[1]

Design Element	Guidelines for Current ADT		
	0 – 399	400 -1,499	1,500 or more
Design Speed (mph)[2]	30	30	40
Shoulder Width (ft)	0	1	3
Lane Width (ft)	10	11	11

Table 4-2: 3R Minimum Design Guidelines for Rural Two-Lane Highways[1]

Design Element	Guidelines for Current ADT		
	0 – 399	400 -1,499	1,500 or more
Surfaced Roadway Width (ft)	20	24	28
Turn Lane Width (ft)[3]	10	10	10
Clear Zone (ft)[4]	7	7	10
Bridges: Width to be retained (ft)[5]	20	24	24[6]

Notes:
1. These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area or to provide operational improvements at specific locations.
2. Considerations in selecting design speeds for the project should include the roadway alignment characteristics as discussed in this chapter. Projects should be designed to target as close to the anticipated posted speed as practical.
3. For two-way left turn lanes, 11-ft to 14-ft usual.
4. A clear zone is the unobstructed, traversable area provided beyond the edge of the through traveled way for the recovery of errant vehicles. The clear zone is measured from the edge of travel lane to the obstruction. If a high design speed (50 mph or greater) is selected for project, the clear zone is 16-ft.
5. Where structures are to be modified, bridges should meet approach roadway width as a minimum. (Approach roadway width is the total width of the lanes and shoulders). Greater bridge widths may be appropriate if the rehabilitation project increases roadway life significantly or if higher design values are selected for the remainder of the project. Existing structure widths less than those shown may be retained if the total lane width is not reduced across or near the structure.
6. For current ADT exceeding 2,000, minimum width of bridge to be retained is 28-ft.

Table 4-3: 3R Minimum Design Guidelines for Urban Streets All Functional Classes[1]

Design Element	Guideline
Design Speed (mph)[2]	30
Lane Width (ft)	10
Turn Lane Width (ft)[3]	10
Parallel Parking Lane Width (ft)	7
Curb Offset for Curbed Streets (ft)	0
Shoulders for uncurbed streets (ft)[4]	2
Clear Zone (ft)	To back of curb or outside edge of shoulder
Bridges: Width to be retained (ft)	Approach roadway, not including shoulders

Table 4-3: 3R Minimum Design Guidelines for Urban Streets All Functional Classes[1]

Design Element	Guideline
Notes: 1. These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area or to provide operational improvements at specific locations. 2. Considerations in selecting design speeds for the project should include the roadway alignment characteristics as discussed in this chapter. Projects should be designed to target as close to the anticipated posted speed as practical. 3. For two-way left turn lanes, 11-ft to 14-ft usual. 4. Minimally 1-ft of shoulder surfaced where lane width is 10-ft thereby providing a 22-ft surfacing width.	

Design Values

Where existing highway features comply with the design values given in this chapter, the designer may choose not to modify these features. However, where existing features do not meet these values, the designer should upgrade to the values shown in this chapter. These values are intended for use on typical non-freeway rehabilitation projects. The designer may select higher values to be consistent with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area, or to provide operational improvements at specific locations.

Alignment

Typically, 3R projects will involve minor or no change in vertical or horizontal alignment. However, consider flattening of curves or other improvements where a crash history indicates a concern or where existing curvature is inconsistent with prevailing conditions within the project or on similar roadways in the area. Where appropriate, consider superelevation improvements as well.

These types of isolated improvements are not considered substantial and thus may be included for consideration in 3R projects. Substantial changes in existing horizontal and/or vertical alignment improvements are considered reconstruction. Projects with substantial changes in existing alignment should be developed to reconstruction (4R) standards.

Design exceptions or design waivers for vertical or horizontal alignment on a 3R project will only be required when crash history or prevailing conditions indicate needed upgrades, but those upgrades are not included in the 3R project. See Chapter 1 Section 2 Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight Network (THFN) Design Deviations for specific geometric criteria required for a design exception or design waiver.

Design Speed

Reconstruction of horizontal and vertical alignments should be considered when the design speed of the roadway in question is not consistent with the existing geometrics. For rehabilitation purposes, the minimum design speed for rural multi-lane highways is 50 mph. The minimum design speed for high-volume rural two-lane highways and high-volume rural frontage roads is 40 mph. The minimum design speed for low-volume rural two-lane highways, low-volume rural frontage roads, urban streets, and urban frontage roads is 30 mph.

For roadways that do not meet the minimum 3R design speeds, an evaluation should be done to examine high-frequency crash locations (and potential crash locations) to determine whether cost-effective alignment revisions can be accomplished with the resources available. These Projects should be designed to accommodate the posted speed where practical. When the posted speed cannot be met, additional traffic safety measures should be provided including, but not limited to, enhanced signage, pavement markings and delineation.

Side and Backslopes

Existing side and backslopes should be retained except where crown widening, grade changes, or not meeting clear zone requirements create conditions that dictate otherwise.

Lane Widths

Consideration should be given to increasing lane widths to 12-ft in conjunction with rehabilitation projects where the highway is a high-volume route utilized extensively by large trucks. Widening shoulders has generally been shown to have a greater reduction in crashes versus widening travel lanes. Factor in this consideration along with other considerations that determine the scope of a project, including expected service life of the proposed rehabilitation work, long-range plans for the route, and design standards of nearby segments on the route.

Section 3 — Designing for Safety

Overview

Resurfacing, restoration, and rehabilitation projects must identify and incorporate appropriate safety enhancements. Use engineering judgement to determine the extent to which safety improvements can reasonably be made with the limited resources available. Crash history and traffic volumes are important factors to consider when evaluating cost-effectiveness of potential safety improvements. Typically, safety improvements are the most cost-effective on roadways with higher traffic volumes. This should not imply that safety enhancements on lower traffic volume roadways are not to be considered. Low-cost safety enhancements can reduce crash frequency and/or severity on all roadways.

Safety Design

TRB's Special Report 214, Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation, describes a safety conscious design process for 3R projects as follows:

> "Significant improvements in safety are not automatic by-products of RRR projects; safety must be systematically engineered into each project. To do this, highway designers must deliberately seek safety opportunities specific to each project and apply sound safety and traffic engineering principles. Highway agencies must strengthen safety considerations at each major step in the design process, treating safety as an integral part of design and not as a secondary objective. These actions require that highway agencies devote greater resources to RRR project design. . ."

Special Report 214 recommends considering project specifics early in the 3R design process. These suggestions are paraphrased as follows:

- At the beginning of 3R project design, highway designers should assess existing physical and operational conditions related to safety.
  - Gather data to identify specific safety problems that might be corrected and compare this data with the system-wide performance of similar highways. This can be conducted by gathering Crash Records Information System (CRIS) reports on the project location for at least 3 previous years. Use the DRIS Comprehensive Safety Dashboard and CRIS Crash Trees to identify both targeted locations and systemic safety countermeasures;
  - Conduct a hot spot analysis of crash data to determine if safety problems arise because of an animal vehicle collision;
  - Conduct a site inspection using experienced personnel to recognize opportunities for safety improvements within the common operating conditions of that individual roadway. This could include carcass data collection details (either within TxDOT or by a county or

city) to determine if safety issues may be arising, as this is a natural pathway for wildlife movement; and

- Determine and verify existing geometry such as roadway widths, horizontal and vertical curvature, superelevation, stopping sight distance restrictions, location and design of intersections, side slopes, clear zone recovery distances, and other geometrics specific to the roadway section being examined.

◆ In addition to pavement repairs and geometric improvements, designers of 3R projects should incorporate other intersection, roadside, and traffic control improvements that enhance safety, including wildlife crossing structures.

- Evaluate less costly safety measures at horizontal curves where reconstruction cannot be accomplished such as widening narrow pavements, flattening steep side slopes, and removing or relocating roadside obstacles. Additionally, designers can consider High Friction Surface Treatment (HFST) to address deficiencies in curve radius or superelevation if good life-cycle cost benefits are determined;
- Evaluate whether TMUTCD requirements are met and, if not, ensure the proposed design will meet the requirements; and
- Routinely evaluate guard fence installations at bridge approaches, and existing bridge rails for rehabilitation or replacement. Include approach signing or delineation if appropriate, regardless of whether bridge widening is necessary on a particular project.

◆ Before developing construction plans and specifications, designers should document the project evaluation, and the design criteria which will be used. Other methods have been successfully used to identify potential safety problems. These methods may be used at the designer's option to meet the particular needs of the project.

◆ Partner with maintenance personnel, local law enforcement, and EMS who are familiar with a particular route and can point out problem areas to the designer based on their experiences. These individuals frequently “work” crash locations and are called upon to perform corrective work necessitated by crashes. Carcass data collection information can be used to determine if there are natural pathways for wildlife movement.

◆ Conduct a crash analysis. Refer to Chapter 5 of the *Highway Safety Manual* (HSM) for diagnosis procedures to identify causes of collision, safety concerns, and crash patterns. Coordinate with Design Division-Traffic Simulation and Safety Analysis Section for further guidance. Additional information is available from the Traffic Safety Division. Coordinate with District Traffic personnel for more information on traffic safety and operational improvements. District Traffic personnel have the expertise to suggest corrective safety countermeasures that should be designed into the 3R project.

◆ Carcass data collection information can be used to determine if there are natural pathways for wildlife movement. Run a hot spot analysis of crash data to determine factors for safety problems, including vehicle crashes caused by animals.

- Before developing the construction plans and specifications, consult the 4-year District Safety Plan and applicable Strategic Highway Safety Plan (SHSP) strategies. A summary of the safety and operations evaluation should be included in the project files and be made available during plan review. This evaluation should document the presence, or absence, of any major deficiencies which may contribute to operational issues and frequency and/or severity. Use this evaluation when scoping work so corrective measures may be taken where practicable.

For needed Project Documentation information see Section 7.

Basic Safety Improvements

Basic safety improvements will be required for all 3R projects. Basic safety improvements are defined as:

- Upgrading guard fence to current standards;
- Providing signing and pavement markings in accordance with the Texas MUTCD and the Traffic Safety Division's Traffic Engineering Standard Sheets;
- Providing a skid resistant surface; and
- Safety treating cross drainage pipe culverts 36 inches in diameter or smaller that are inside the clear zones given in this chapter.

Other safety improvements to consider include treatment of nonstandard mailbox supports, nonstandard luminaire supports, and nonstandard sign supports that are inside the suggested clear zones. Trees, utility poles, or other obstacles that are indicated significantly in a crash evaluation should also be considered.

Guard Fence

Guard fence must be upgraded to current hardware standards. Connections to structures, post spacing, and end treatments must meet current design practices. Where guard fence height is 3 or more inches higher or lower than criteria, height corrections are required. Guard fence lengths will generally be designed to requirements given in Appendix A, Determining Length of Need of Barrier.

Remove all unneeded guard fence and guard fence where shielded obstacles may be cost-effectively design treated (removed, made yielding, etc.).

Headwalls

Remove headwalls on small (36 inches or less) cross drainage pipe culverts that are inside the clear zones given in this chapter and install sloping (1V:3H or flatter) culvert ends that blend with existing side slopes. Where located behind guard fence, these culvert ends should be safety treated and the guard fence should be removed where there are no other obstructions involved.

Other Safety Improvements

Cross Drainage Culverts

Cross drainage box and pipe culverts greater than 36-in may remain as they exist where the clear zones given in this chapter are satisfied. Where the clear zones given in this chapter are not met, safety treatment (grates, extension, or guard fence) will be required. In situations where the culvert end meets clear zone requirements, yet other culverts within the project limits are treated, consider safety treatment to provide consistency within the project area. Where guard fence is required for shielding other non-removable obstacles, headwalls behind guard fence do not need to be safety treated.

For culvert spans from 3-ft to 5-ft and heights up to 5-ft that need to be safety treated, pipe grated design is very effective from a safety standpoint and generally cost effective from an economic standpoint. If sloping or grated inlet designs are utilized for these low height and width culverts and their past performance has not been satisfactory, then evaluate inlet restrictions (entrance loss coefficients) for their effects on hydraulics. If necessary, reference TxDOT's *Hydraulic Design Manual* for entrance loss coefficients with various configurations as well as other hydraulic design information.

The ends of bridge-class pipes and culverts must be protected regardless of clear zone, see Chapter 2, Section 7 for additional guidance.

Driveway Embankments and Pipes

Treat driveway embankments and pipes on 3R projects only where other design improvements necessitate their reconstruction or when they are located inside the clear zones given in this chapter.

The extent of the safety improvement selected for a particular project may be influenced by the extent of other work. Where pavement improvements extend pavement life substantially, consider more significant geometric and safety related improvements.

Section 4 — Frontage Roads

Overview

Table 4-4 and Table 4-5 show geometric design guidelines for 3R projects on rural and urban freeway frontage roads. These guidelines are acceptable for those projects involving either rehabilitation of only the frontage road or rehabilitation of the frontage road in conjunction with rehabilitation of the freeway mainlanes. 3R frontage road design guidelines should not be used when a freeway section is reconstructed from right-of-way line to right-of-way line, even though no additional frontage road lanes are added. Complete frontage road reconstruction projects should reference the applicable reconstruction guidelines for the appropriate criteria.

Frontage roads are built in some locations initially in a phased construction sequence with mainlanes to be built when traffic conditions warrant. If the frontage road is serving as the principal roadway pending future mainlane construction, apply the 3R design guidelines for rural multilane highways for rehabilitation work on these facilities.

Table 4-4: 3R Design Guidelines for Rural Frontage Roads[1]

Design Element	Guideline for Current ADT	
	0 - 1,499	**1500 or more**
Design Speed (mph)	30	40
Lane Width (ft)	10	11
Shoulder Width (ft) Two-Way Operation	1	3
Inside Shoulder Width (ft) One-Way Operation	1	2
Outside Shoulder Width (ft) One-Way Operation	1	4
Clear Zone (ft)	7[3]	16
Bridges[2]: Width to be retained(ft)	24	24[4]

Table 4-4: 3R Design Guidelines for Rural Frontage Roads[1]

Design Element	Guideline for Current ADT	
	0 - 1,499	**1500 or more**

Notes:

1. These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area, or to provide operational improvements at specific locations.
2. Where structures are to be modified, bridges should meet approach roadway width as a minimum. (Approach roadway width is the total width of the lanes and shoulders.) Greater bridge widths may be appropriate if the rehabilitation project increases roadway life significantly or if higher design values are selected for the remainder of the project. Existing structure widths less than those shown may be retained if the total lane width is not reduced across or near the structure.
3. For high-speed design (50-mph or greater), minimum clear zone is 16-ft.
4. For current ADT exceeding 2,000, minimum width of bridge to be retained is 28-ft.

Table 4-5: 3R Design Guidelines for Urban Frontage Roads[1]

Design Element	Guideline for All Traffic Volumes
Design Speed (mph)	30
Lane Width (ft)	10
Shoulder Width[2] (ft)	0 inside 2 outside
Clear Zone (ft)	Back of curb or outside edge of shoulder
Curb Offset (ft)	1 either side
Bridges: Width to be retained (ft)	Approach roadway not including shoulders

Notes:

1. These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area, or to provide operational improvements at specific locations.
2. For uncurbed shoulder widths see Table 4-4.

Section 5 — Bridges, Including Bridge-Classification Culverts

Overview

Where minimum bridge widths exist, it is generally expected that no additional structural work will be necessary. However, existing conditions such as deficient railing (pre-1964 rails are typically in this category), deteriorated deck, or a structure with an unsafe load carrying capability may require additional structural work. In such cases, reference TxDOT's *Bridge Railing Manual* and consult with the Bridge Division for design recommendations. If structural modification is necessary, it may be appropriate to consider a greater bridge width if future plans or traffic projections indicate additional roadway improvement will be necessary in the foreseeable future.

To accomplish a complete and cost effective rehabilitation plan throughout a geographic area, identify roadways with low traffic volumes and perform a crash evaluation on structures with railings that do not match current standard railing details (reference TxDOT's *Bridge Railing Manual* and *Bridge Railing Standards*). Evaluate bridges with these railings on an individual basis. If the evaluation indicates continuing satisfactory performance and the railing is in good repair, retain these railings on low volume roadways.

Obtain additional information on bridge rehabilitation in TxDOT's *Bridge Project Development Manual*.

Note that additional vertical clearance requirements may apply to bridge projects on the Texas Highway Freight Network (THFN) as specified in Chapter 3, Section 8.

Section 6 — Super 2 Highways

Overview

A Super 2 highway is where a periodic passing lane is added to a two-lane rural highway to allow slower vehicles to pass and traffic platoons to disperse. The passing lane will alternate from one direction of travel to the other within a section of roadway allowing passing opportunities in both directions. A Super 2 project can be introduced on an existing two-lane roadway where there is a significant amount of slow moving traffic, limited sight distance for passing, prevalence of head-on crashes, and/or the existing traffic volume has exceeded the two-lane highway capacity (creating the need for vehicles to pass on a more frequent basis).

Widening of the existing pavement can be symmetric about the centerline or on one side of the roadway depending on right-of-way availability and ease of construction.

Some issues to consider when designing a Super 2 project:

- Analyze existing right-of-way width considerations to determine feasibility of upgrading to a Super 2;
- Consider providing a left turn or right turn lane if a significant traffic generator falls within the limits of a Super 2;
- Consider providing wider shoulders (8-ft to 10-ft) in areas with high driveway density;
- Evaluate the location and associated treatment to achieve clear zone values at large drainage structures and bridges when considering the placement of passing lanes;
- Evaluate traffic operations including truck volumes if consideration is given to terminating passing lanes on significant uphill grades. Coordinate passing lanes with climbing lane needs to improve operating characteristics;
- Avoid terminating a passing lane over a hill or around a horizontal curve where the pavement surface at the end of the taper isn't visible from the beginning of the taper;
- Consider traffic operations, unexpected lane changes, and intersection geometrics when evaluating the termination of a passing lane at an intersection. If termination of the passing lane at the intersection would result in significant operational lane weaving, then consider extending the passing lane beyond the intersection;
- Provide adequate sight distance (stopping sight distance desirable) between the end of a lane termination taper and a constraint such as metal beam guard fence, a narrow structure, or major traffic generator; and
- Consider providing the passing lane in the direction leaving an incorporated area for potential platoons generated in the urban area.

Basic Design Criteria

Recommended design values are shown in Table 4-6.

Table 4-6: Design Criteria for Super 2 Highway

	Minimum	Desirable
Design Speed (mph)	See Table 4-2	See Table 4-1
Clear Zone (ft)	See Table 4-2	See Table 4-1
Lane Width (ft)	11	12
Shoulder Width (ft)	3[1]	8 - 10
Passing Lane Length (mi)	1	1.5 - 2[2]
Notes: 1. Where ROW is limited. 2. Longer passing lanes are acceptable, but more than 4-mi are not recommended. Consider switching the direction if more than 4-mi.		

The length for opening a passing lane (Figure 4-1) should be based on the following:

$$L = \frac{WS}{2}$$

The taper length for closing a passing lane (Figure 4-1) should be based on:

$$L = WS$$

Where

L = Length of taper (ft),

W = Lane width (ft), and

S = Posted speed (mph).

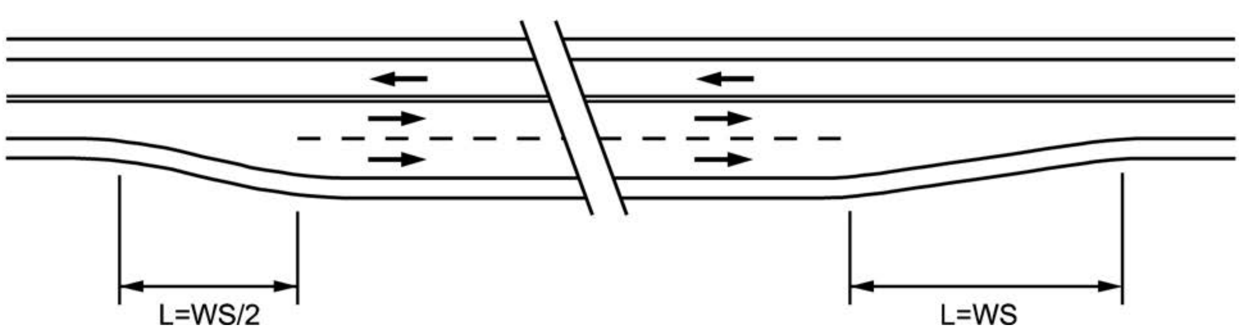


Figure 4-1. Opening and Closing an Outside Passing Lane

When switching the passing lane from one direction to another (closing the passing lane in each direction), provide a taper length from each direction based on L = WS, with a minimum 50-ft buffer (stopping sight distance (SSD) desirable) between them. (Figure 4-2).

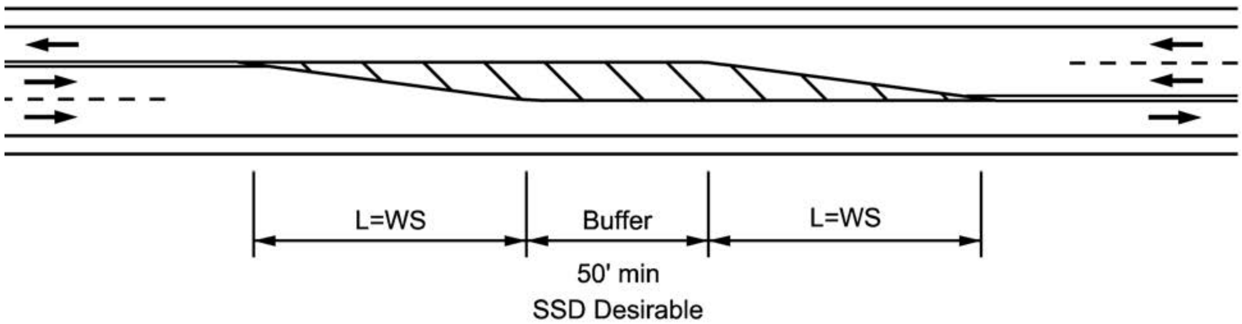


Figure 4-2. Closing an Inside Passing Lane

When opening a passing lane in each direction (Figure 4-3), provide a taper length based on L= WS/2.

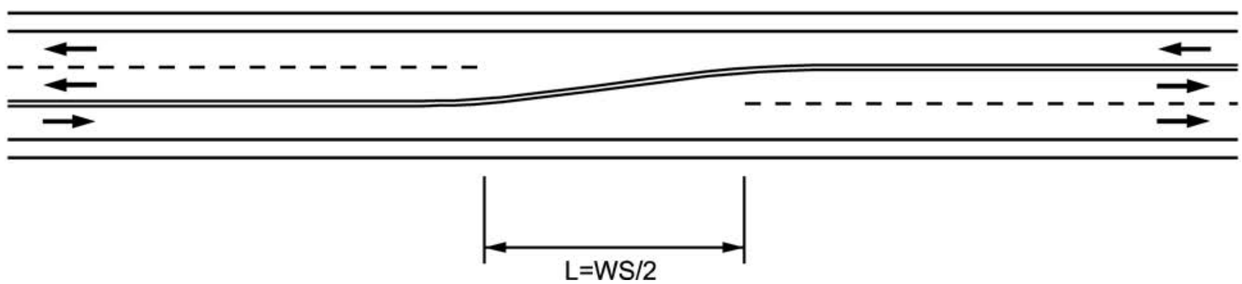


Figure 4-3. Opening an Inside Passing Lane

When widening to the outside of the roadway to provide a passing lane opportunity (Figure 4-4), provide an opening taper length based on L = WS/2 and a closing taper length based on L=WS.

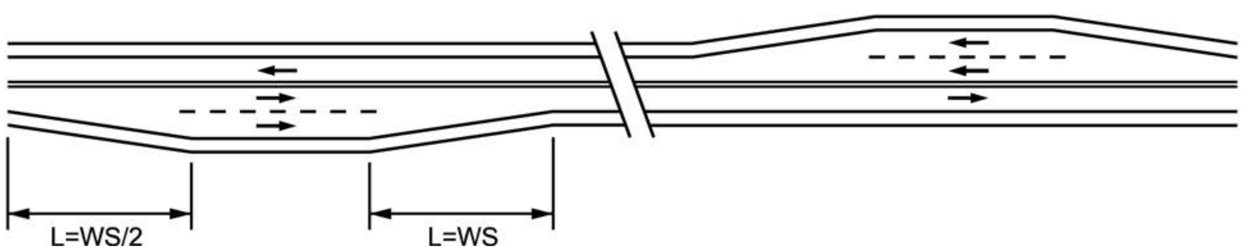


Figure 4-4. Separated Passing Lanes with an Outside Widening

Passing lanes in each direction may overlap if ROW is sufficient (Figure 4-5).

Provide an opening taper length based on L = WS/2 and a closing taper length based on L=WS.

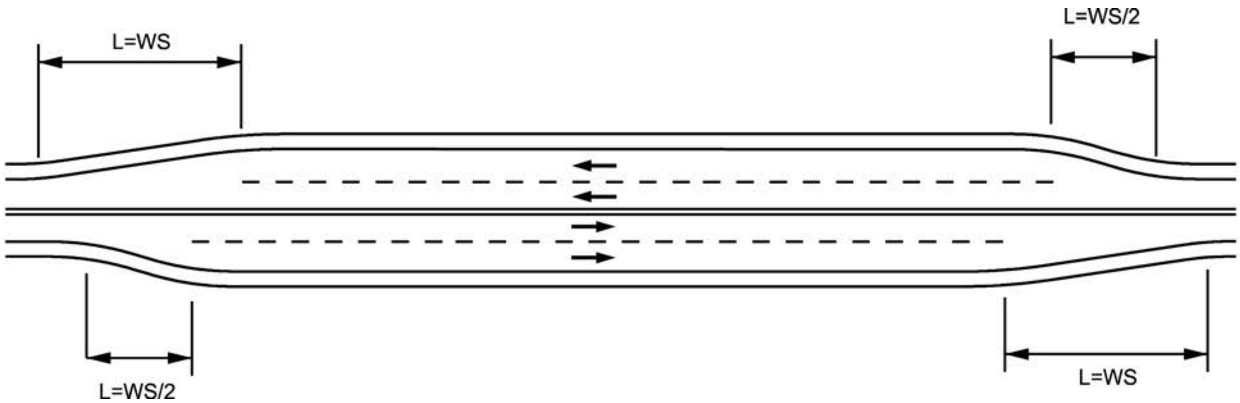


Figure 4-5. Side by Side Passing Lanes

Section 7 — 3R Project Documentation

Project-Specific Design Information

The Project-Specific Design Information has been developed to assist in the project evaluation and provide one possible outline for file documentation.

For individual project evaluation, consider:

- Has an on-site evaluation of the project been conducted (date, time, personnel)?
- What is the highway type (low volume two-lane, urban street, etc.)?
- What design guidelines given in this chapter are applicable to this project?
- What are the design values on the existing roadway?
- What are the expected design values of the roadway after project completion? Which design elements require individual evaluation prior to final design?
- What is the ADT and character (truck %, recreational use, local traffic, etc.) of the traffic using the roadway?
- What is the crash history (minimum 3 years including type, severity, conditions, etc.) of the entire project and at any specific locations that require the individual evaluation of design elements?
- What is the compatibility of the proposed design with adjacent sections of the roadway?

For specific design elements that require individual evaluation prior to final project design, consider:

- What length and percentage of the project are affected by the design elements in question?
- What is the comparative cost of the given design guideline versus the proposed design element in terms of construction, right-of-way availability, project delay, environmental impacts, etc.?
- What is the long-term effect of using the design element selected in terms of capacity and level of service?

If other design elements required individual evaluation, what is believed to be the cumulative effect of these design elements on the safety and operation of the proposed facility?

Plan Set Requirements

The following, at a minimum, should be included with all 3R PS&E submissions:

- Design speed for the project (same as shown on page 3 of form 1002)

- Existing and proposed typical sections showing lane and shoulder widths, obstruction clearances, and bridge widths
- Existing and proposed information on bridge rails and the structural capacity of all structures
- Plan and profile sheets detailing the proposed horizontal alignment, superelevation, and vertical alignment if the existing grade or horizontal alignment are changed at specific locations. If the scope of work is only overlaying, widening shoulders, or other simpler types of work, where the existing grade line is basically being followed,
- Summary sheet (tabular presentation) showing horizontal curve data and vertical cure data from previous plans may be used. The plans could also be a combination of proposed realignment plan & profile sheets at specific locations and a summary sheet of existing geometry for the remainder of the project.
- Existing and proposed information on safety appurtenances (such as guard fence and safety end treatments)

The summary sheet indicated above should be signed and sealed by the responsible engineer with a note documenting that the appropriate engineering analysis was accomplished. The note will vary depending on the analysis completed. The following is an example of a note that might be used:

> "Existing bridge rails are <type of rail> constructed in <year> and structures meet HS <rating> loading. This project meets the basic safety requirements of the 3R design criteria. Guard fence (including connection to structures, post spacing and end treatments), signing, and pavement markings meet current standards. Cross drainage culverts, parallel culverts, mailbox supports, luminaire supports, and sign supports within the required clear zone of <distance> feet have been safety treated or are being upgraded to standards."

Chapter 5 — Non-Freeway Resurfacing or Restoration Projects (2R)

Contents:

Section 1 — Criteria

Overview

This section provides design guidance for 2R projects meeting all the following conditions:

- Project is on a non-freeway facility;
- Not on the National Highway System (NHS); and
- Current ADT ≤ 2,500 per lane.

These guidelines should also be used in determining design scope and estimating cost for individual candidate projects whenever a restoration program is being developed. Preliminary structural planning should be coordinated with the Bridge Division.

Definition

Restoration (2R) projects are defined as work performed to restore pavement structure, riding quality, or other necessary components, to their existing cross section configuration. The principal purposes of these projects are surfacing and repair of the pavement structure. The addition of through travel lanes is not permitted under a 2R project. The addition of continuous two-way left-turn lanes (TWLTL), acceleration/deceleration lanes, turning lanes, and shoulders are acceptable as restoration work as long as the existing through lane and shoulder widths are maintained as a minimum. The restoration work may include upgrading roadway components as needed to maintain the roadway in an acceptable condition.

If bicycle accommodations are provided, they must meet the requirements specified in Chapter 6, Section 4 (Bicycle Facilities).

Upgrading

Where the work is cost effective and funds are sufficient to upgrade to reconstruction or rehabilitation design criteria without jeopardizing district priorities for other restoration work, development of projects to higher criteria may be done at the District’s discretion.

Crash Analysis

A crash analysis (minimum of 3 years) must be conducted for 2R projects. Any specific areas involving high crash frequencies will be reviewed and corrective measures taken where appropriate in the current project or another project already in the planning stages. In addition to a formal analysis of crash data, Chapter 4, Section 3, Safety Enhancements lists several methods that have been used successfully to identify potential crash problems.

Chapter 6 — Special Facilities

Contents:

Section 1 — Off-System Bridge Replacement and Rehabilitation Projects

Overview

This section provides design guidance for projects meeting all the following conditions:

- Included in the off-system bridge replacement and rehabilitation program;
- Facility not likely to be added to the designated state highway system; and
- Current ADT of 400 or less.

If all the above conditions are not met, then the design criteria for the appropriate functional class of highway should be utilized. For off-system bridge projects, current ADT may be used with the appropriate functional class of highway (i.e., enter tables, charts, or figures with current ADT substituted for future ADT). Where significant traffic growth is expected or the roadway will be widened in the near future, the use of future ADT for design purposes is encouraged.

For more information on the off-system bridge replacement and rehabilitation program, refer to TxDOT's *Bridge Project Development Manual*.

Additional vertical clearance requirements may apply to bridge projects on the Texas Highway Freight Network (THFN) as specified in Chapter 3, Section 8.

Design Values

Design values selected for a particular project must satisfy and preferably exceed the values shown below. Selected design values should be consistent and compatible with the prevalent design features on the existing off-system roadway. If the route has the potential for significant ADT increases in the near future, or if the character of the traffic is not local, design requirements for the appropriate class of highway must be used.

- Minimum Design Speed: Meet or improve conditions that are typical on the remainder of the roadway.
- Vertical Curvature, Minimum K values: Meet or improve conditions that are typical on the remainder of the roadway.
- Horizontal Curvature: Meet or improve conditions that are typical on the remainder of the roadway.
- Minimum Superelevation: Meet or improve conditions that are typical on the remainder of the roadway.
- Maximum Grades: Meet or improve conditions that are typical on the remainder of the roadway.

- Minimum Structure Width, Face to Face of Rail: 24-ft. for approach roadway widths of 24-ft. or less. 28-ft. is the minimum if the approach roadway is over 24-ft. in width, or the county or municipal entity has near term plans to widen the approach roadway, or anticipates near term development.
- Bridge End Guard Fence:
  - Minimum Conditions – Transition consistent with current applicable Roadway Design Standards.
  - If an intervening roadway or driveway prevents usual placement of guard fence, a guard fence radius may be used consistent with specific application in the applicable Roadway Design Standard.
- Approach Roadway:
  - For minimum length of 50-ft adjacent to the bridge end, the roadway crown should match clear width across structure plus additional width to accommodate approach guard fence.
  - An appropriate transition (minimum length 50-ft) to county road width should be made in the sections of approach roadway located at the federal project extremities.
  - If roadway surfacing is included, a minimum of the bridge roadway width should be used for the 50-ft roadway section adjacent to the bridge.
- Traffic Control:
  - When provided, and to the extent feasible, detours should at a minimum match existing county road design features. Design details for detours should be shown in the plans and on the preliminary layouts.
  - Traffic control devices should be in conformance with the *Texas MUTCD*, and details should be included in the plans.

Section 2 — Historically Significant Bridge Projects

Reference for Procedures

Historically significant bridges command importance and a place in the engineering and cultural heritage of this nation. Federal law requires these bridges be given special consideration, where practical and feasible, toward their preservation in the course of bridge replacement or bridge rehabilitation/improvement projects.

Reference can be made to TxDOT's *Historic Bridge Manual* for procedures that should be used when developing projects that involve historic bridges.

Section 3 — Texas Parks and Wildlife Department (Park Road (PR) and Park and Wildlife Road (PW)) Projects

Working Agreements

According to Acts 1995, 74th Leg., Ch. 445, §1, the Texas Department of Transportation must construct, repair, and maintain roads in and adjacent to state parks, state fish hatcheries, state wildlife management areas, and support facilities for parks, fish hatcheries, and wildlife management areas.

In response to this legislation, a memorandum of agreement between TxDOT and the Texas Parks and Wildlife Department (TPWD) was established. This memorandum of agreement states that TPWD is to provide TxDOT with current design standards for TPWD facilities.

Accordingly, TPWD facilities, which are those designated as Park and Wildlife Roads (PW), are to be designed based on the criteria and guidance given in the current publication of the *Texas Parks and Wildlife Department Design Standards for Roads and Parking*.

Park Roads (PR) that lead to or enter a state park, and are designated on the State Highway System, are to be designed based on the criteria and guidance given in the current RDM. For roads designated as PR where constrained site conditions, parking, or high pedestrian usage warrant a lower speed; a design speed of no lower than 20 mph may be used.

Section 4 — Bicycle Facilities

This section discusses the features and design criteria for bicycle facilities and includes the following subsections:

- 6.4.1 General
- 6.4.2 Planning and Context
- 6.4.3 Elements of Design
- 6.4.4 Bicycle Facility Types
- 6.4.5 Intersections and Crossings
- 6.4.6 Maintenance, Operations, and Work Zone
- 6.4.7 References

6.4.1 General

6.4.1.1 Purpose

This chapter provides guidance on the design of bikeways with the goal of accommodating people of all ages and abilities riding bicycles. The application of this guidance will apply to TxDOT roadways and any project funded by TxDOT.

Note that “colored-pavement markings” are not currently prescribed in TxDOT policy. For the purposes of this guidance, in pictures, and figures where colored markings are shown they are for depiction purposes only to illustrate the general location of bicycle traffic. Additionally, the signing and pavement markings shown are examples. Current standard signing and pavement markings may be found in the TMUTCD, Standard Highway Sign Designs for Texas (SHSD) and applicable Traffic Standard Sheets.

6.4.1.2 Definitions

The following definitions are provided for the purpose of this Guide; therefore, definitions may vary when reviewing other sources.

- Bikeways: Any road, path, or facility intended for bicycle travel which designates space for bicyclists distinct from motor vehicle traffic, such as bike lanes, buffered bike lanes, bike accessible shoulders, shared use paths, and separated bike lanes. A bikeway does not include shared lanes, sidewalks, signed routes, or shared lanes with shared lane markings.
- Bicycle Facilities: A general term denoting provisions to accommodate or encourage bicycling, including bikeways, bicycle detection, shared lanes and shared lane markings, wayfinding, as well as parking and storage facilities.

6.4.1.3 Relationship to Other Policies, Laws, and Regulations

6.4.1.3.1 AASHTO and FHWA Guidelines

This TxDOT Bicycle Accommodation Design Guidance is based on the review of national guidelines for the best practices for the design of bicycle facilities and is the governing bicycle guidance document for TxDOT. The 2012 AASHTO Guide for the Development of Bicycle Facilities (AASHTO Bike Guide) continues to be the governing document for specific design criteria that is not contained within this TxDOT Bicycle Accommodation Design Guidance. However, for guidance not contained within this TxDOT Bicycle Design Guidance, the FHWA Bikeway Selection Guide should be considered a companion to the AASHTO Bike Guide, and in instances of contradictions the FHWA guide shall take precedence because it contains design guidance more current than the AASHTO Bike Guide. The 2012 AASHTO Bike Guide does not cover certain bikeway types. For example, design for separated bike lanes is quickly evolving and, as such, a flexible design approach is encouraged. For further information on FHWA's position on design flexibility, refer to the August 2013 memo "*Bicycle and Pedestrian Facility Design Flexibility.*"

6.4.1.3.2 U.S. Department of Transportation Policy

On March 11, 2010, U.S. DOT signed a federal policy statement on Bicycle and Pedestrian Accommodations Regulations and Recommendations. This policy statement emphasized that, "every transportation agency, including a state DOT, has the responsibility to improve conditions and opportunities for walking and bicycling." The statement encourages transportation agencies "to go beyond minimum standards to provide safe and convenient facilities for these modes" on all transportation projects.

6.4.1.3.3 National Statutes

Under 23 U.S. Code 217(g)(1) it states, "Bicycle transportation facilities and pedestrian walkways shall be considered, where appropriate, in conjunction with all new construction and reconstruction of transportation facilities, except where bicycle and pedestrian use are not permitted."

6.4.1.3.4 Texas Codes

The Texas Administrative Code (TAC) and the Texas Transportation Code (TTC) provide directives for the design of bicycle facilities in the state of Texas. Title 6 §201.902(c) of the TTC requires TxDOT to adopt rules relating bicycle use on the roads in the state highway system. Title 43 §25.53 of the TAC specifies that TxDOT must take bicycle accommodation into consideration during the planning and implementation of all construction and rehabilitation projects. Title 43 §25.54 of the TAC specifies that TxDOT will adopt the latest version of the AASHTO *Bike Guide* and will continue to review guidelines for design, construction, and maintenance of bicycle facilities with the intent to adopt new guidelines as appropriate.

Accordingly, this TxDOT Bicycle Accommodation Design Guidance is based on the review of national guidelines for the best practices for the design of bicycle facilities and supersedes aspects of the AASHTO *Bike Guide*. If a specific design criterion or guidance is not provided herein, see Section 6.4.1.3.1 for document precedence. It is anticipated that all guidance herein provides further enhancements to the safety and comfort level of most cyclists compared to the 2012 AASHTO *Bike Guide*.

6.4.1.4 Projects that can be Excepted from Bicycle Accommodations

Bikeways should be routinely included when planning and designing transportation facilities, addressing the needs of the target design user (see Section 6.4.2.3); consequently all projects must be consistent with the identified needs for bikeways as identified in the NEPA process. Exceptions to providing bikeways are permitted if the project meets one or more of the following criteria. Although an area may fall under one of the exceptions below, it is important to plan for anticipated growth where bicycling activity might become more prevalent in the future during the life of the project. MPO and local planning documents should be reviewed and coordinated with to identify anticipated future growth when selecting bikeways outside the urbanized boundaries. The documentation for having an exception based on the following criteria will be maintained with the project file with specific documentation as to the nature of the exception, but this is not considered a formal Design Exception or Design Waiver. The circumstances requiring a formal Design Exception or Design Waiver are documented in Section 6.4.1.6 of this guidance.

Note, projects located on the Texas Bicycle Tourism Trails Example Network are not excepted from providing bikeways regardless of location. The TxDOT Statewide Planning Map provides additional information on MPO boundaries, area types, and the Texas Bicycle Tourism Trails Example Network. Additionally, all On-System bridges regardless of location, involving bridge replacement, bridge deck replacement, or bridge rehabilitation will need to meet the bicycle clear space requirements specified in Section 6.4.2.4, and are not excepted. Off-system Bridges, with current ADT greater than 400 ADT, where this addition may represent an unreasonable increase in cost may be excepted from the bicycle clear space requirement, see Ch. 6, Section 1 for specific off-system bridge requirements for current ADT of 400 or less.

1. The project is on a roadway where bicycle travel is specifically prohibited by law or Texas Transportation Commission Minute Order.
2. The project is located outside of a respective Metropolitan Planning Organization (MPO) Boundary; AND is also located outside of any respective city limits with a population of 2,500 or greater. The TxDOT Statewide Planning Map provides additional information on MPO boundaries and area types. Before using this exception, project designers should seek out and consider local stakeholder input and community need.
3. The project is in an urbanized setting (defined as a city, town, or Census-designated place with a population of 2,500 or greater) where a locally preferred alternative route has been adopted or implemented and bikeways are deemed impractical within the scope of the project. The project is in an urbanized setting with limited roadway improvements, and there is already a

future project programmed (e.g., MPO Active Transportation Plan) where the bicycle updates would make more sense in the context of overall transportation improvements.

4. The cost to provide features exclusively for bikeways is excessively disproportionate to the need or likely uses. While a determination of "excessively disproportionate" should be concluded on a case-by-case basis and well documented, exceeding 20% of the total project cost (including design, construction, ROW, etc.) may be considered as a general guideline. This exception should not be used if the project will help complete a gap in an overall bicycle network.

5. The source of funding specifically precludes improvements other than those for which the funding is intended. Note that although Category 8 funding (which includes HSIP, Statewide systemic widening, and Road to Zero) does not currently have funding allocated specifically for bikeways, it is allowable to place money that has been specifically designated for bicycle accommodations into Category 8. Note, the following link from FHWA provides funding opportunities for bicycle facilities. (https://www.fhwa.dot.gov/environment/bicycle_pedestrian/funding/funding_opportunities.cfm)

6. The type of work is limited in scope such that major roadway elements are not being constructed or reconstructed. For example: safety end treating culverts only, Metal Beam Guard Fence (MBGF) replacement only, sealcoat only, and other types of preventative maintenance projects. Note that resurfacing can provide the opportunity to restripe and/or improve the riding surface for bikeways in certain instances and, as such, would not necessarily warrant an exemption. Other projects with a narrow scope should be evaluated to determine if negative impacts to the bikeway may result.

6.4.1.5 Use of Dimensional Values for Bikeway Design

Due to the fact bicyclists operate with or adjacent to motor vehicles, the provision of facilities which meet minimum dimensions alone may not always ensure a safe or comfortable bikeway for bicycle travel. Because bicyclists are often operating with motor vehicles, designers should consider the bicyclists' perception of safety and the type of bikeway relative to the context in which the bikeway is located. In many instances the use of minimum values for design criteria do not account for the user's perception of safety using the facility. The perception of how safe a person feels on the transportation system can have significant impacts on how they choose to use or avoid the facilities provided. Assessments of perceived safety for the same site will vary between observers but is increasingly measurable by comfort rating tools found in the Highway Capacity Manual. Perceived safety is analogous to "subjective" safety as defined by the AASHTO *Highway Safety Manual*.

The following terms are used throughout the guide to define the desirable, minimum, and constrained conditions for which bikeway widths will be determined:

6.4.1.5.1 Desirable Values

Desirable values are stated explicitly throughout the chapter by using the words "desirable" or "desired." In many instances, these will be presented as a range of values (e.g. bike lane width). The design value should be chosen to meet the purpose and need objectives of the bicycle facility where practicable. In general, desirable values (typically larger values) should be used to maximize the safety and comfort benefits for bicyclists and other users. Alternative values should only be used in locations where it is not possible to use desirable values due to social, economic, and environmental impacts.

6.4.1.5.2 Minimum Values

Minimum values are either implied by the lowest value in a range, or explicitly stated throughout the chapter by using the word "minimum." The use of minimum values should not automatically be considered a default for bikeways due to the inherent vulnerability of bicyclists in the event of a crash. In some instances, the use of minimum design values may result in trade-offs with respect to the comfort and safety of bicyclists.

6.4.1.5.3 Constrained Values

Where the use of a minimum design value may degrade bicyclist safety or comfort, the words "in constrained conditions" are used. In general, the use of constrained values should only be considered:

- for limited distances (such as to bypass a transit stop or to accommodate a bikeway on an existing bridge);
- as an interim measure where the larger values will result in the preferred design not being constructible; or
- at locations with low volumes of bicyclists where those volumes are anticipated to remain low.
- Additional engineering countermeasures at locations should be considered where the use of constrained values is likely to increase crash risk or reduce bicyclist comfort.

6.4.1.6 Design Exceptions and Waivers for Dimensional Values of Bicycle Facilities

As previously discussed, the process for deciding if projects can be excepted from providing bicycle facilities does not require a design exception or design waiver. However, if a project includes bicycle facilities and the preferred bicycle facility type cannot meet the respective criteria or thresholds, designers can select the next best facility type following the guidance in Section 6.4.2.5. A design exception or design waiver is needed if the minimum criteria for the selected facility is not met.

6.4.1.6.1 Urbanized Context (Urban Core/Urban/Suburban/Rural Town)

Design Exceptions

Bike Lane: If the minimum width specified in the Basic Design Guidelines is not met.

Shared Lane (Wide Outside Lane): If the traffic volume, speed, or width criteria (14-ft maximum, 13-ft minimum) specified in the Basic Design Guidelines are not met.

Design Waivers

Shared Use Path (Independent alignment or sidepath): If the minimum width criteria (minimum 10-ft, 8-ft constrained), buffer width, and other geometric criteria specified in the Basic Design Guidelines, and the associated AASHTO Bike Design criteria are not met.

Separated Bike Lane/Buffered Bike Lane: If the minimum criteria specified in the Basic Design Guidelines are not met.

6.4.1.6.2 Rural Context

Design Exceptions

Shared Lane (Wide Outside Lane): If the traffic volume, speed, or width criteria (14-ft maximum, 13-ft minimum) specified in the Basic Design Guidelines are not met.

Design Waivers

Shared Use Path (Independent alignment or sidepath): If the minimum width criteria (minimum 10-ft, 8-ft constrained), buffer width, and other geometric criteria specified in the Basic Design Guidelines, and the associated AASHTO Bike Design criteria are not met.

Bike Accessible Shoulder: For new construction, reconstruction, or widening projects in a rural setting where right-of-way is being acquired, a Design Waiver is required if a minimum width defined in the Basic Design Guidelines for each bikeway type is not provided (see Section 6.4.4).

6.4.2 Planning and Context

6.4.2.1 Bicycle Planning Principles

Effective bikeway design and network planning often leads to more people bicycling by creating routes that are efficient, seamless, and easy to use. Having a clear understanding of good planning and design principles is important as these concepts will ultimately be the foundation for the design intention and potential trade-offs that may occur. Bicycle planning principles that can be used include safety, comfort, connectivity, and cohesiveness. Descriptions of these principles are provided in Table 6-1.

Table 6-1. Bicycle Planning Principles

Safety	◆ Separate bicycles from motorized traffic where the speed differential and traffic volumes are higher. ◆ Reduce conflicts between bicycles and pedestrian traffic on shared use paths (sidepaths). ◆ Minimize speed differential at conflict points where practical to minimize or eliminate injury. ◆ Provide sufficient clearances to obstacles to avoid crashes. ◆ Reduce or eliminate conflicts along the route including intersecting roads and driveways.
Comfort	◆ Recognize that different bicycle users have varying levels of comfort for various roadway conditions. ◆ Minimize exposure to traffic, noise, and emissions. ◆ Minimize or avoid conditions that require bicyclists to dismount during a trip. ◆ Provide sufficient shy distance to obstacles. ◆ Establish geometric criteria that provide a comfortable facility to operate a bicycle. ◆ Minimize or avoid conflicts with pedestrian traffic.
Connectivity	◆ Accommodate local bicycle and transit transportation routes and networks. ◆ Connect bikeways and intersecting streets at a local scale for access to destinations. ◆ Allow for user choice of routes by providing a dense and connected network. ◆ Provide seamless transitions between different on-road and off-road facility types. ◆ Eliminate barriers and provide continuous bikeways to support network connectivity. ◆ Integrate design with local bicycle transportation plans. ◆ Carry bikeways through intersections of on-system roads with off-system roads.
Cohesiveness	◆ Employ a direct and logical structure that minimizes turns and promotes staying on the network. ◆ Inform all roadway users clearly of the presence of bicyclists, especially at conflict points. ◆ Provide clear and intuitive transitions between different yet connected bicycle facility types. ◆ Extend bikeways to logical and safe termini.

6.4.2.2 Context Considerations

Context and engineering judgment play important roles in selecting the appropriate bicycle accommodations. FHWA identifies four components that are important in identifying what type of bicycle accommodation to use: project limits, land use context, types of bicyclists the bicycle accommodation is expected to serve, and key safety and performance criteria.

As part of these overarching themes, the elements outlined below should be documented in the design process and used to determine the selection of an appropriate bicycle facility type, as discussed in the following sub-sections:

- Project Identification: Project name, project ID (CSJ), roadway name, limits, county;

- Roadway Context: Adjacent roadway functional class, speed, average daily traffic volume, project length, intersection frequency and crossing road functional classification, driveway density;
- Area Context: Land use context (see below);
- Intended Bicycle Accommodation Users: Target design user (interested but concerned or all ages and abilities); and
- Other Roadway Users: Truck percentage and key movements, transit operation (headway) and key stops, curbside lane activity, expected pedestrian demand.

6.4.2.2.1 Land Use Contexts

The land use context that surrounds a potential bikeway may influence the type of users (e.g. target design user), the number of users, and the potential interactions of other roadway users with the facility. Two context groupings have been used when providing guidance for bikeway selection:

- Urban and Suburban Contexts (referred to as "urbanized" and includes urban core and rural town which is defined in FHWA's Separated Bike Lane Planning and Design Guide); and
- Rural Contexts.

The urban core and rural town are land use contexts that are anticipated to be added to a future version of the Roadway Design Manual that will allow additional flexibility with respect to roadway, and bicycle facility planning and design. The urban core would be contained within the current definition of an urban area. The rural town would be contained within the current definition of a rural area. A city with a minimum population of 2,500 and less than a population of 50,000 is defined as an urbanized cluster by the US Census and for the purposes of this guidance will fall within the Urban and Suburban Contexts. Note that certain towns with populations less than 2500 will have characteristics of a rural town as described below and may utilize the rural town bicycle criteria as deemed applicable.

Urban Core

The urban core context includes areas of the high-density, with mixed land uses among predominantly high-rise structures and with small building setbacks. The urban core context is found predominantly in central business districts and adjoining portions of major metropolitan areas. On-street parking is often more limited and time restricted than in the urban context. Substantial parking is in multi-level structures attached to or integrated with other structures. The area is accessible to automobiles, commercial delivery vehicles, and public transit. Sidewalks are present nearly continuously, with pedestrian plazas and multi-level pedestrian bridges connecting commercial parking structures in some locations. Transit corridors, including bus and rail transit, are typically common and major transit terminals may be present. Some government services are available, while other commercial uses predominate, including financial and legal services. Structures may have multiple uses and setbacks are not as generous as in the surrounding urban area. Residences are often apartments or condominiums. Driver speed expectations are low and pedestrian and bicycle flows are high. See Figure 6-1 for a depiction of Urban Core.

Source: Gresham-Smith Partners

Figure 6-1. Typical Street in the Urban Core Context.

Rural Town

The rural town context applies to roads in rural areas located within developed communities. Rural towns generally have low development densities with diverse land uses, on-street parking, and sidewalks in some locations, and small building setbacks. Rural towns may include residential neighborhoods, schools, industrial facilities, and commercial main street business districts, each of which present differing design challenges and differing levels of pedestrian and bicycle activity. The rural town context recognizes that rural highways change character where they enter a small town, or other rural community, and that design should meet the needs of not only through travelers, but also the residents of the community. See Figure 6-2 for a depiction of Rural Town.

Source: Gresham-Smith Partners

Figure 6-2. Typical Street in the Rural Town Context.

6.4.2.2.2 Speed and Volumes of Motor Vehicles

There are many factors to consider when selecting and designing bikeways, with motor vehicle speed and volume as the initial determinants of suitable bicycle facilities. The influence of speed and volume on the safety and perceived safety or comfort of bicycle riders is an important factor and the respective criteria for appropriate speeds and traffic volumes is contained in the subsequent guidance.

For the purposes of this bicycle guidance when a speed criteria is mentioned it will mean the higher of the design or posted speed (speed limit). The vehicular ADT or traffic volumes referenced pertain to existing conditions. The respective sidepath, or bicycle volumes referenced pertain to existing conditions or anticipated beginning conditions. Note that the anticipated growth in usage should also be considered when defining the footprint for the bicycle accommodations.

6.4.2.2.3 Other Factors

Other factors that should be considered in the selection of bikeways are listed in the *FHWA Bikeway Selection Guide* and summarized in Table 6-2.

Table 6-2. Other Factors to Consider in Selection of a Bikeway

Factor	Description and Design Considerations
Unusually high motor vehicle peak hour volumes	On roadways that regularly experience unusually high peak hour volumes, more separation can be beneficial, particularly when the peak hour also coincides with peak volumes of bicyclists.
Traffic vehicle mix	Additional separation between bicyclists and motorists is particularly important on moderate volume to high-volume streets where heavy vehicles are an abnormally high percentage of traffic. Higher percentages of trucks and buses increase risks and discomfort for bicyclists due to vehicle size and weight, and the potential for motorists to not see bicyclists due to blind spots. This is particularly a concern for right turns, where large vehicles may appear to be proceeding straight or even turning left as they position to make a wide right turn movement. Visibility and awareness of bicyclists can be improved by providing: ◆ additional buffer width between a separated bike lane or shared use sidepath to the travel lane; ◆ providing markings and signs denoting the crossing; ◆ providing raised crossings; or ◆ at signalized locations, phase separating the conflict.

Table 6-2. Other Factors to Consider in Selection of a Bikeway

Factor	Description and Design Considerations
Parking turnover and curbside activity	Parked or temporarily stopped motor vehicles present a risk to bicyclists high parking turnover and curbside loading (commercial and passenger) may expose bicyclists to being struck by opening vehicle doors or people walking in their travel path. Vehicles stopped within bicycle lanes or travel lanes may require bicyclists to merge into an adjacent travel lane. In locations with high parking turnover, or curbside loading needs, wider bike lanes or separated bike lanes in lieu of bike lanes, can help to alleviate conflicts. This issue also encompasses locations where transit vehicles load and unload passengers within a bicycle lane or shared curb lane.
Driveways/intersection frequency	The frequency of driveways and intersections also impacts decisions regarding the design of separation between the street and the bicycle accommodation as well as the design of driveways. Motorists need adequate sight distance to enter and exit intersections and driveways and benefit from sufficient space to yield to bicyclists. This is particularly important for sidepaths (the AASHTO Bike Guide enumerates the potential areas of conflict) and two-way separated bike lanes located on one side of two-way streets where contra-flow bicyclist may be unexpected by motorists. High driveway frequency may make a one-way bicycle facility type a preferable option. Consideration may be given to consolidating driveways as applicable. Wider buffers and clear sight lines can improve bicyclist safety. Where contra-flow bicycling occurs, additional design features that slow motorists' turning movements and give motorists more time to see oncoming bicyclists may significantly improve safety for all users. Frequent, closely spaced driveways may limit the ability to provide vertical elements necessary to provide separated bike lanes. In these locations, buffered bicycle lanes, bicycle lanes or shoulders may be the only viable bicycle facility unless it is feasible to provide a raised bike lane at sidewalk level to provide greater separation from traffic.
Direction of operation	For separated bikeways, a determination must be made as to whether the bikeway will be provided as a one-way facility on each side of the road, a two-way facility on one side of the road, or as two-way facilities on both sides of the road. As discussed above, the contra-flow bicyclist may be unexpected by motorists requiring additional design mitigations. This decision requires engineering judgment based on the bikeway's role in the broader bike network, connectivity, safety impacts, the locations of destinations within the corridor, physical constraints within the ROW, and an assessment of intersection operations and frequency of driveways and intersections.
Vulnerable populations	The presence of high concentrations of children and older adults should be considered during project planning. These groups may only feel comfortable bicycling on physically separated facilities, even where motor vehicle speeds and volumes are relatively low. Typically, these populations are less confident in their bicycling abilities and, in the case of children, may be less visible to motorists and lack both roadway experience as well as sufficient cognitive or physical maturity to recognize and anticipate potential conflicts. They can also create more conflicts with pedestrians when they are expected to share the same space.

Table 6-2. Other Factors to Consider in Selection of a Bikeway

Factor	Description and Design Considerations
Network connectivity gaps	It is essential to consider the proposed transportation project in context of the local bicycle network. Wide, high-volume, or high-speed roadways can create substantial barriers to connectivity. Parallel alternative routes may not exist or may require bicyclists to ride several miles out of the way, adding substantial distance and travel time. Intersections between state and local roadways may feature a high number of conflict points, constrained right-of-way, or high-speed differential. Separated facilities can help close gaps in a low-stress network. Considerations include providing separate bicycle facilities under freeway underpasses, improving visibility of bicyclists, providing on-street connections between two major shared use paths, or routes connected to schools, major employers, parks, or other recreational opportunities.
Transit considerations	Biking offers a valuable “first-mile” and “last-mile” connection to transit systems, effectively expanding the transit-shed around a station or stop. It is important to ensure accessibility of transit boarding areas, pedestrian crossings, and parking spaces, while also integrating the bicycle network into transit systems. Traffic laws and agency policy often address transit vehicles and bicycles in the right most lane or right side of the roadway. Some agencies have designated shared “transit lanes” for bicycle riding, but frequent bus stops or roadway design may create delays or less safe conditions for bicyclists sharing a lane with heavy transit traffic. If the preferred bikeway for a roadway is a bike lane or separated bike lane, the placement of the bike lane with respect to where pedestrians may wait or travel when boarding or alighting transit vehicles should be considered, as should the extent to which transit operations impact bicyclists’ level of comfort and safety. As noted in FHWA’s *Separated Bike Lane Planning and Design Guide*, options for minimizing conflicts with transit include installing signs, pavement markings, and/or floating bus stops to provide for shared space, placing a separated bike lane on the left side of a one-way street (out of the way of transit stops along the right side), or choosing to install a separated bike lane on a nearby parallel corridor away from transit.

6.4.2.3 Target Design User

Different bicycle riders may have varying tolerances associated with the importance of the individual planning principles outlined above. Figure 6-3 indicates the array of potential bicycle riders that should be considered when scoping and designing a roadway project.

Figure 6-3. Types of Bicycle Facility Users as a Percentage of Total General Population

Interested but Concerned	Somewhat Confident	Highly Confident
(51 – 56%)	(5 – 9%)	(4 – 7%)

Research over the last decade has evaluated how to classify the general population into different types of bicyclists. This research confirms that only a relatively small percentage of people can be classified as comfortable bicyclists in mixed traffic, and that a large majority of people prefer some level of separation from higher-volume, higher-speed motorized traffic. As such, in many jurisdictions across the United States, the common target design user are those who are interested in riding but concerned about safety ("Interested but Concerned" bicyclists) as this is the largest group of potential bikeway users among the general population. According to research, these bicyclists would ride more if they felt safer and, thus, are more likely to take short trips, avoiding busier arterial roadways. "Interested but Concerned" bicyclists prefer separation from vehicles and have a lower tolerance for traffic stress than more confident riders.

To maximize the potential for bicycling as a viable transportation option, it is important to design facilities to meet the needs of the "Interested but Concerned" bicyclist user profile. Bicycle facilities which meet the needs of the "Interested but Concerned" bicyclist will generally meet the needs of all bicyclists, therefore they are considered "All Ages and Abilities" bicycle facilities because they maximize potential use. One exception to this are situations where bicyclists are operating in large groups, or where individual bicyclist who prefer to operate at higher speeds (typically greater than 20 mph) who may prefer to operate in the travel lane. In general, more separation from motorized traffic is desirable to serve a greater number and type of users more safely. TxDOT endeavors to provide bicycle facilities to serve bicyclists of "All Ages and Abilities" to maximize the number of people who may use the facility.

6.4.2.4 General Bikeway Selection

The figures in this section provide some general guidance for the bicycle planning principles, target design user, and context discussed in the previous sections. Note, these Figures are for very general guidance and a preliminary bikeway recommendation. Section 6.4.4 provides specific design criteria and additional guidance for the application of bikeways to serve a target design user. In particular, elements associated with the respective roadway speed criteria discussed in the application discussion or each bikeway in Section 6.4.4 should be reviewed before making a final bikeway selection and choosing specific design elements. As long as the project meets the defined criteria in Section 6.4.4 for a specific bikeway, the engineer has the discretion to select the bikeway that is best suited to the project needs and constraints.

Figure 6-4 provides an example selection process from FHWA. Note that Figures 6-5 and 6-6 should be used to make the initial recommendation for the type of bicycle facility.

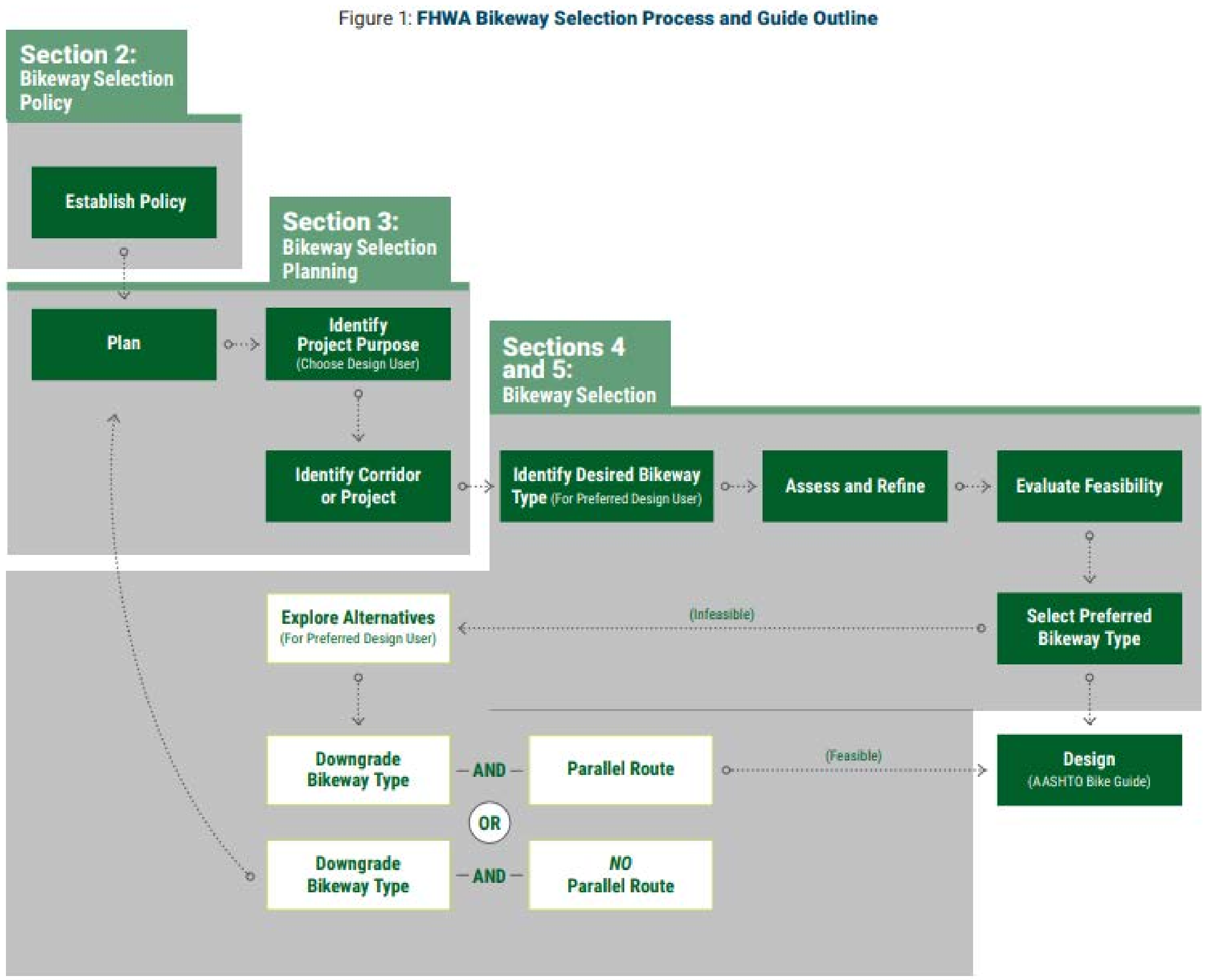


Figure 6-4. FHWA Bikeway Selection Process and Guide Outline

6.4.2.4.1 Requirements for Selection – Urban, Urban Core, Suburban, and Rural Town

For all urbanized (urban, urban core, suburban, and rural town) contexts, the following guidance is provided for various types of construction (the selection guidance is based primarily on FHWA's *Bikeway Selection Guide*):

- For full reconstruction projects, new construction projects, or other construction projects whether in existing right-of-way or with additional right-of-way, the project should use Figure 6-5 for the initial recommendation for bikeway facility type. See section 6.4.2.5 for guidance on downgrading to the next best facility type. For the selected facility type, the geometric values should adhere to the specific guidance for that facility type. For values not provided in this guide, refer to the AASHTO *Guide for the Development of Bicycle Facilities* and, for separated bike lanes, FHWA's *Separated Bike Lane Planning and Design Guide*. Note: TAS/ADAAG requirements for each facility type must be met as well.

- For projects involving bridge replacement, bridge deck replacement, or bridge rehabilitation, the following guidance is provided:
  - A 5-ft minimum clear space (4-ft shoulder and 1-ft offset measured to the toe of barrier) shall be provided on the structure and along the adjacent barrier. Off-System Bridges, with current ADT greater than 400 ADT, where this addition may represent an unreasonable increase in cost may be excepted from this requirement. See Ch. 6, Section 1 for specific off-system bridge requirements for current ADT of 400 or less. Where feasible, desirable shoulder width as shown in Figure 6-6 should be used.
  - For roadways identified on the Texas Bicycle Tourism Trails Example Network, preferred 10-ft (minimum 8-ft) shoulder width should be provided on bridges.

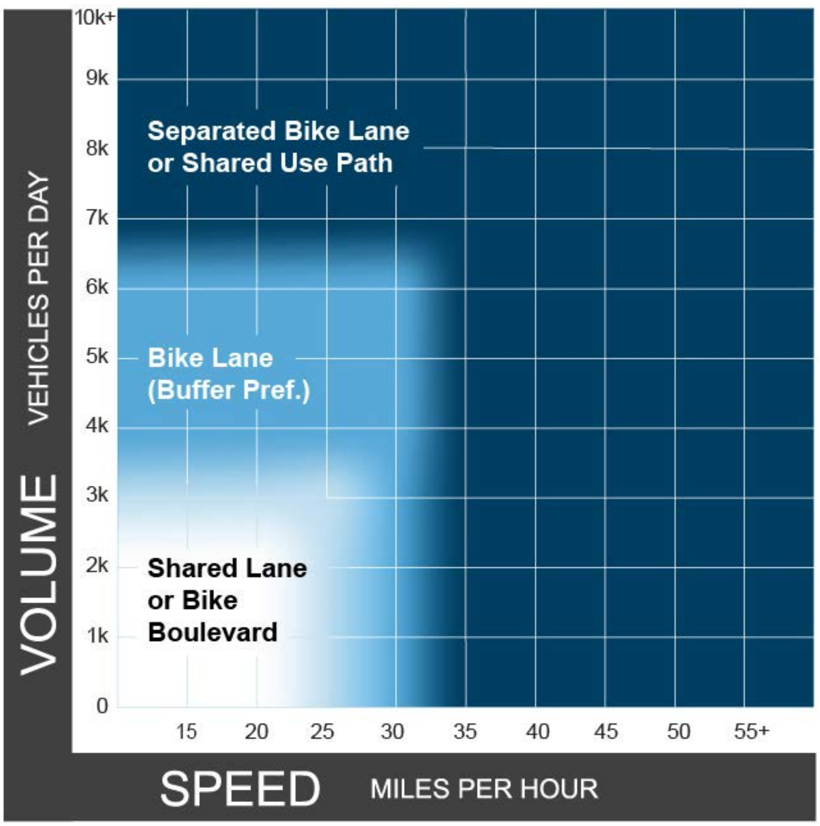


Figure 6-5. Recommended Bicycle Facility Selection for Urban, Urban Core, Suburban, and Rural Town Context

NOTE: Use the higher of the design speed and the posted speed for the speed. To be conservative when designing for all ages and abilities facilities, designers should attempt to use the higher-level facility at the respective boundary limits.

6.4.2.4.2 Requirements for Selection – Rural

For rural context (not rural towns), the below guidance is provided for various types of construction:

- For new construction, reconstruction, or widening projects in a rural setting where right-of-way is being acquired, the following guidance is provided:
  - When the scoping process and NEPA documentation indicates a need for bicyclist accommodations, the recommended bikeway is shown in Figure 6-6, indicating the desirable shoulder widths for various speeds and traffic volumes. In some cases, a shared use path or other locally preferred facility type may be identified during stakeholder outreach. See Section 6.4.4.9 for additional guidance on shoulders and shared lanes. See section 6.4.2.5 for guidance on downgrading to the next best facility type. Note: TAS/ADAAG requirements for each facility type must be met as well.
  - Roadways indicated in TxDOT's Bicycle Tourism Trails Study should be designed with a minimum 8-ft shoulder, a shared use path, or another locally preferred facility type.
- Where new construction, reconstruction, or widening is accomplished without additional right-of-way, the above should be followed where feasible. See section 6.4.2.5 for guidance on downgrading to the next best facility type. Note: TAS/ADAAG requirements for each facility type must be met as well.
- For projects involving bridge replacement, bridge deck replacement, or bridge rehabilitation, the following guidance is provided:
  - A 5-ft minimum bicycle clear space (4-ft shoulder and 1-ft offset measured to the toe of barrier) shall be provided on the structure and along the adjacent barrier. Off-system Bridges, with current ADT greater than 400 ADT, where this addition may represent an unreasonable increase in cost may be excepted from this requirement. See Ch. 6, Section 1 for specific off-system bridge requirements for current ADT of 400 or less. Where feasible, desirable shoulder width as shown in Figure 6-6 should be used.
  - For roadways identified on the Texas Bicycle Tourism Trails Example Network, preferred 10-ft (minimum 8-ft) shoulder width should be provided on bridges.

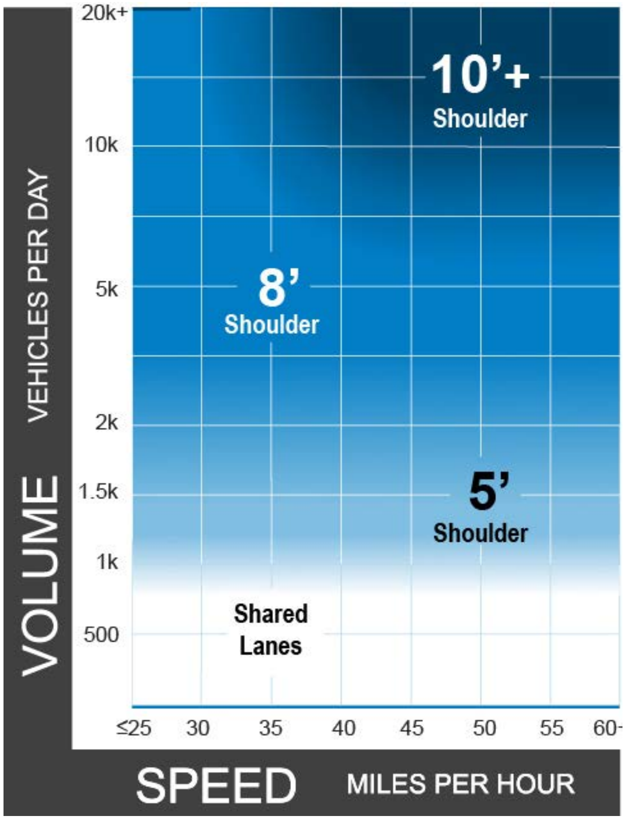


Figure 6-6. Recommended Bicycle Facility Selection for Rural Context

NOTE: A separated shared use path is a suitable alternative to providing paved shoulders solely for the purpose of a bikeway and should be considered on Bicycle Tourism Trail Example Network segments as well as rural roads with ADT above 6000 vehicles per day. Use the higher of the design speed and the posted speed for the speed. If the percentage of heavy vehicles is greater than 5%, consider providing a wider shoulder or a separated path.

6.4.2.5 Bikeway Feasibility Assessment

There will be locations, mostly due to ROW not being acquired, where desirable or minimum bikeway width(s) are not feasible, even after all other design criteria were analyzed as minimum values (e.g. lane widths). In these cases, it will be necessary to consider downgrading the bikeway to the next best facility type and/or to provide a parallel facility. The following should be considered when evaluating bikeway feasibility.

6.4.2.5.1 Prioritizing Safety

When evaluating safety trade-offs, options that reduce serious injuries and fatalities should be prioritized over options that may reduce property damage or minor injuries.

Meeting safety and mobility goals are typical objectives for roadway designers. Designers have an ethical obligation to provide for the health, safety, and welfare of the public, which may require a careful evaluation of mobility goals where they have the potential to degrade safety. One user's convenience or mobility should not be prioritized over another user's safety. Most roadway and bikeway design projects can be designed to improve safety for all modes.

As discussed in Section 6.4.1.5, the dimensional values of bicycle facilities influence the safety and comfort of bicyclists. The impact of the use of minimum and constrained dimensional values should be considered carefully during feasibility assessments.

When a bike lane approaches a large intersection, several decisions have to be made that can impact safety. In locations with constrained right-of-way, the designer may have to evaluate whether to terminate a bikeway to provide an additional travel lane or turn lane. Intersections are locations where a high percentage of bicyclist crashes occur. It is preferable to maintain the bikeway to maximize safety and comfort of bicyclists. If the bikeway cannot be continued, care should be taken to ensure the transition is clear and consideration should be given to providing bicyclists an option to leave the roadway (see Section 6.4.5.2 for further guidance.)

Bikeways should remain consistent along a corridor. However, it may be preferable for safety reasons to increase separation as a bicycle facility approaches an intersection – for example a shoulder could transition to bicycle lane, or a bicycle lane could transition to a separated bicycle lane with a protected intersection at large intersections to minimize conflicts between motorists and bicyclists.

6.4.2.5.2 Considerations for Alternatives

Impacts on ridership, comfort/stress, safety, and overall network connectivity should be considered when evaluating alternative bikeway designs or parallel routes to ensure the project will still meet the purpose identified at the outset. The following tradeoffs need to be considered and documented in the design process:

- Reduced or suppressed ridership where the bikeway does not meet the needs of the target design user;
- Additional length of trip when bicyclists must use a parallel route. This length should not exceed 30% more than original route;
- Failure to provide a bikeway or critical connections that leave an important gap in the bicycle network;
- Reduced safety where bicyclists must operate with relatively high motor vehicle speed and/or high-volume traffic in shared lanes;
- Reduced safety where bicyclists must operate in narrow bikeways (e.g. narrow bike lanes adjacent to high turnover parking or narrow shared use paths with high volumes of pedestrians or bicyclists);
- Reduced safety where bicyclists improperly use facilities (e.g., ride the wrong way on shared lanes, sidewalk riding, etc.); and
- Increased sidewalk bicycling where bicyclists are avoiding low-comfort/high-stress conditions

In instances where shared use paths or separated bike lanes are recommended by volume, speed, and/or other factors, but desirable facility widths cannot be obtained, it may still be preferable to provide separated facilities with minimum or reduced paths and/or buffer widths rather than putting bicyclists in the roadway with high-speed/volume traffic.

If selecting a parallel route as the preferred route to accommodate a bikeway that meets the needs of the Interested but Concerned bicyclist occurs, the provision of alternative bicycle facilities on the desired route should still be considered to accommodate the Highly Confident design user and to provide connections for bicyclists to and from properties that exist along that desired route. A typical example would be locations where the Interested but Concerned bicyclist is accommodated on a parallel, low volume local street along a bicycle boulevard because there is not sufficient width available to provide a separated bike lane on the desired route. In these instances, the provision of a bike lane or shoulder can be still be beneficial to serve the more confident bicyclists on the higher volume roadway to improve their safety and access to destinations along the roadway.

6.4.3 Elements of Design

6.4.3.1 Design Characteristics of Bicyclists

Regardless of the target design user identified in the bikeway selection process, the adult bicyclist should typically be used to establish geometric design controls because the adult bicyclist is typically the fastest and physically largest user. However, the designer should consider all likely users of a bikeway when establishing design controls. Common exceptions to using the adult bicyclist to establish design controls are:

- Using pedestrian performance criteria at street crossings where pedestrians will be crossing with bicyclists. This is common at shared use path, local street, or bicycle boulevard crossings of arterials, which must be designed to ensure a pedestrian can safely cross the road at a typical walking speed;
- Using the heights and speeds of recumbent bicyclists or child bicyclists for the purposes of establishing sight distances or crossing times at intersections. Both users are shorter in height and slower at starting from a stop compared to adult bicyclists; and
- Using a bicycle with a trailer for the purposes of designing median refuge islands, rail-crossings, or queuing areas because these devices lengthen and widen the operating space required.

6.4.3.2 Bicycle Types

Some of the types of devices that are commonly used on Texas streets and trails are shown below in Figure 6-7. Typical variations in height, width, and length are noted. Although recommended widths for bike lanes, shared use paths, and sidewalks in the RDM generally address these devices, designers should be cognizant of the expected use of the longest and widest devices and provide appropriate accommodations for the use of these devices.

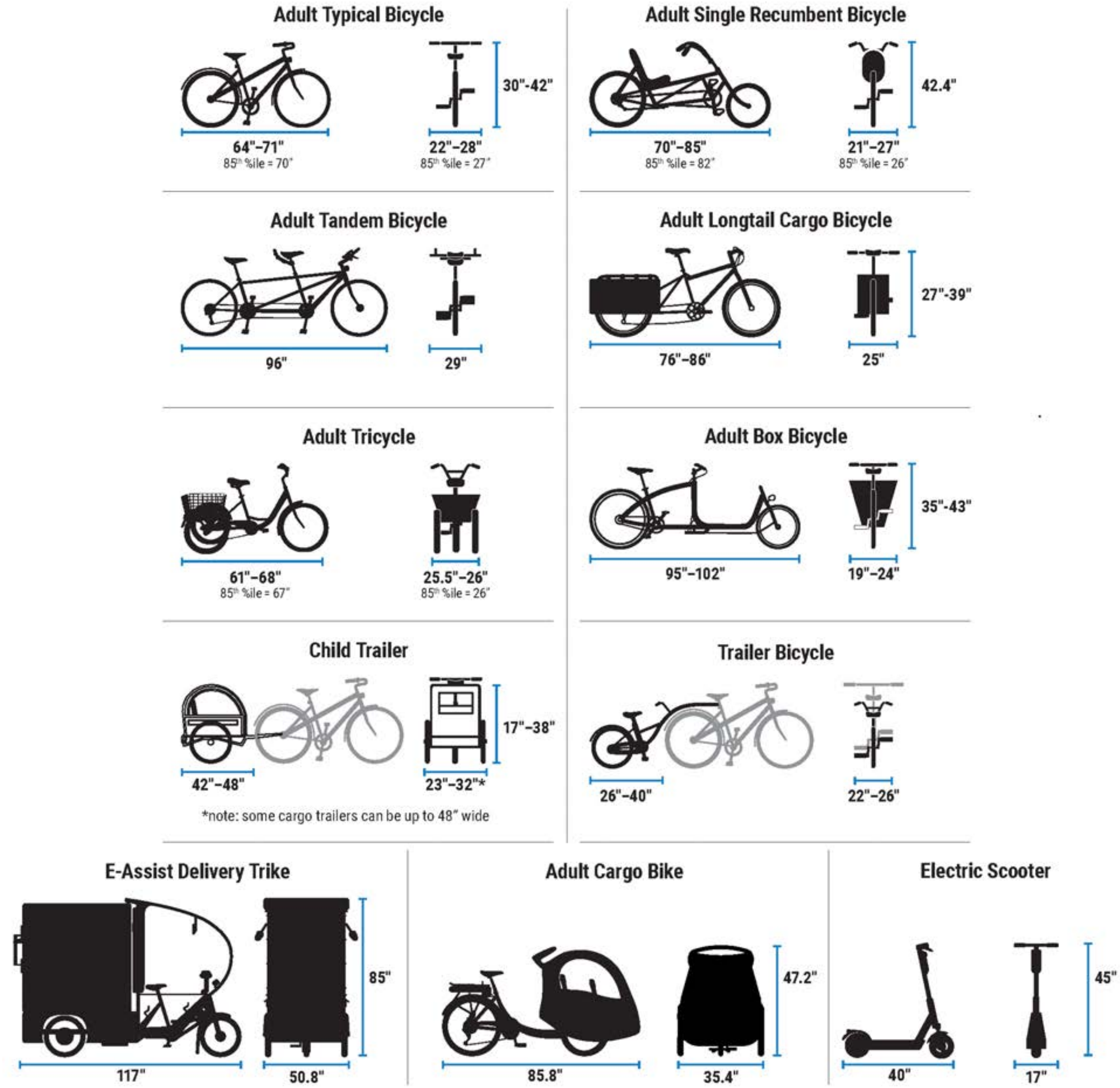


Figure 6-7. Typical Dimensions of Bicycle Types

6.4.3.3 Bicyclist Operating Speeds

The speed of a bicyclist is dependent upon several factors, including the age and physical condition of the user; the type and condition of the user's equipment; the purpose and length of the trip; the condition, location, and grade of the bikeway; the prevailing wind speed and direction; and the number and types of other users on the facility. Although some adults may be able to maintain faster speeds (e.g., 25 to 30 mph) on level grades and attain speeds higher than 35 mph on steep descents, typical adult bicyclists average 8 to 12 mph on flat terrain, and steep inclines may result in speeds comparable to walking (2 to 3 mph). Research has found a median cruising speed for urban bicyclists as 9.7 mph with a 15th percentile speed of 8.2 mph. The 15th percentile speed should be used to inform bicycle crossing speeds at intersections as it represents the lowest range of bicyclist operating speeds.

For this reason, there is no single design speed that is recommended for all bikeways. Design speed considerations for bikeways are incorporated into specific bikeway guidance throughout Section 6.4.4 as appropriate.

6.4.3.4 Sight Distance

Chapter 2, Section 3 of the RDM provides detailed information about motorist Sight Distance. The AASHTO *Bike Guide* provides detailed guidance for evaluating bicyclist stopping sight distance, horizontal sight distance, and the evaluation of sight distance to establish traffic control at trail roadway intersections. The following supplements that guidance.

6.4.3.4.1 Bicycle Stopping Sight Distance

Adequate motor vehicle stopping sight distance is important for the safety of pedestrians and bicyclists who must cross roadways. Refer to Chapter 2 Section 4 Intersection Sight Distance for procedures for determining motor vehicle stopping sight distances.

Bicycle stopping sight distance (SSD) is the distance needed to bring a bicycle to a fully controlled stop. It is a function of the user's perception and brake reaction time, the initial speed, the coefficient of friction between the wheels and the pavement, the braking ability of the user's equipment, and the grade. The coefficient of friction for the typical skidding bicyclist is 0.32 for dry conditions and 0.16 for wet conditions. A perception and brake reaction time of 2.5 seconds should often be used, though 1.5 seconds may be used where braking or stopping is more expected, such when approaching intersections where scanning for conflicts is anticipated.

Minimum Stopping Sight Distance

$$S = \frac{V^2}{30(f \pm G)} + 1.47\,Vt$$

Where:

S	=	stopping sight distance (ft)
V	=	velocity (mph)
f	=	coefficient of friction (0.16 for a typical bike in wet conditions)
G	=	absolute value of grade (ft/ft) (rise/run)
t	=	perception / reaction time (1.5 seconds for expected stops, 2.5 seconds for unexpected stops)

Note: $\pm$ = negative traveling downhill, positive uphill

Stopping Sight Distance (ft) Based on Speed and Grade for a 2.5 Second Perception-Reaction Time

Speed (mph)	Grade (Positive indicates ascending)										
	-10%	-8%	-6%	-4%	-2%	0	2%	4%	6%	8%	10%
10				65	61	58	55	53	52	51	50
11				74	69	66	63	61	59	57	56
12				84	78	74	71	68	66	64	62
15			130	118	109	102	97	93	89	86	84
18	246	201	174	156	143	134	126	120	115	111	108
20	296	240	207	185	169	157	148	140	134	129	
25	440	353	300	266	241	222	208	196	187		
30	611	486	411	361	325	298	277	260			

Figure 6-8. Stopping Sight Distance Formula and Table

6.4.3.4.2 Intersection Sight Distance

Chapter 2 Section 4, Intersection Sight Distance establishes a range of recommended sight triangles that correspond to requirements for motorists to have sufficient space to identify, react, and potentially yield to other traffic at an intersection based on the traffic control applied at the intersection. Applying the sight triangle requirements provided in Chapter 2 Section 4, Intersection Sight Distance will generally result in sufficient sight distance for bicyclists when operating on bicycle facilities located within the street, such as shared lanes, bike lanes, and separated bike lanes.

Intersections of Shared Use Paths and Roadways should be calculated using a combination of Motorists SSD requirements along their travel path and bicyclist SSD along their travel path. The AASHTO *Bike Guide* provides additional discussion on these sight triangles considerations.

6.4.3.4.3 Sight Distance Considerations at Intersections with Separated Bike Lanes and Sidepaths

At intersections with separated bike lanes and sidepaths it is important to evaluate potential right and left turning conflicts across the bikeway with right turning motorists (Case A) and left turning motorists (Case B); both of which create two unique sight triangles. For each case, there are two yielding scenarios based on who is closest to the intersection for which the provision of adequate sight distance between users is paramount:

Turning Motorist Yields to (or Stops For) Through Bicyclists

This scenario occurs when a through moving bicyclist arrives or will arrive at the crossing prior to a turning motorist, who must stop or yield to the bicyclist who is likely to be within the intersection at the time the motorist turns. This is shown visually in the top portion of Figure 6-9. Vertical elements near the intersection to separate bicyclists, including on-street parking, should be set back sufficiently for the motorist to see the approaching bicyclist providing the motorist sufficient time to slow or stop before the conflict point.

Through Bicyclist Yields to (or Stops For) Turning Motorist

This scenario occurs when a turning motorist arrives or will arrive at the crossing prior to a through moving bicyclist. This is shown visually in the bottom portion of Figure 6-9. This scenario can occur when a bicyclist approaches after a motorist has stopped and yielded to other people crossing in the intersection and the crossing is clear for the motorist to proceed. The motorist may begin turning as the bicyclist approaches, requiring the bicyclist to slow and potentially stop while the motorist completes the turning movement.

To provide sufficient sight distance between turning motorists and approaching bicyclist, it is necessary to provide a minimum approach clear space. This clear space is calculated by the determining the amount of space traveled by each user navigating the approach before they reach the conflict point which occurs in each of these zones:

- Recognition zone – the approaching bicyclist, motorist, or pedestrian has an opportunity to see the other user(s) and evaluate their respective approach speeds.
- Decision zone – the bicyclist, motorist, or pedestrian identifies who is likely to arrive at the intersection first and adjusts their speed to yield or stop if necessary.
- Yield/stop zone – a space for the motorist or bicyclist to yield or stop, if necessary.

The key design parameter to determine the necessary approach clear space is the turning speed of the approaching motorist which is influenced strongly by the effective turning radius available to them to execute the turn. It is preferable for turning motorist speeds to be below 15 mph at locations where they are expected to stop or yield to pedestrians and bicyclists in a crossing. Table 6-3 presents approach clear space values assuming the bicyclist design speed is 15 mph and that stopping is occurring under wet conditions and on flat terrain. As both bicyclists and motorists should be expecting potential conflict or the need to stop or yield, reaction times are assumed to be 1.5 seconds. The approach clear space may be increased or decreased to account for other approach speeds or reaction times if they are known.

Table 6-3. Intersection Approach Clear Space by Vehicular Turning Design Speed

Effective Vehicle Turning Radius	Vehicular Turning Speed	Approach Clear Space
<18 ft	<10 mph*	20 ft
18 ft	10 mph	40 ft
25 ft	15 mph	50 ft
30 ft	20 mph	60 ft
≥50 ft	25 mph	70 ft

*most low volume driveways and alleys

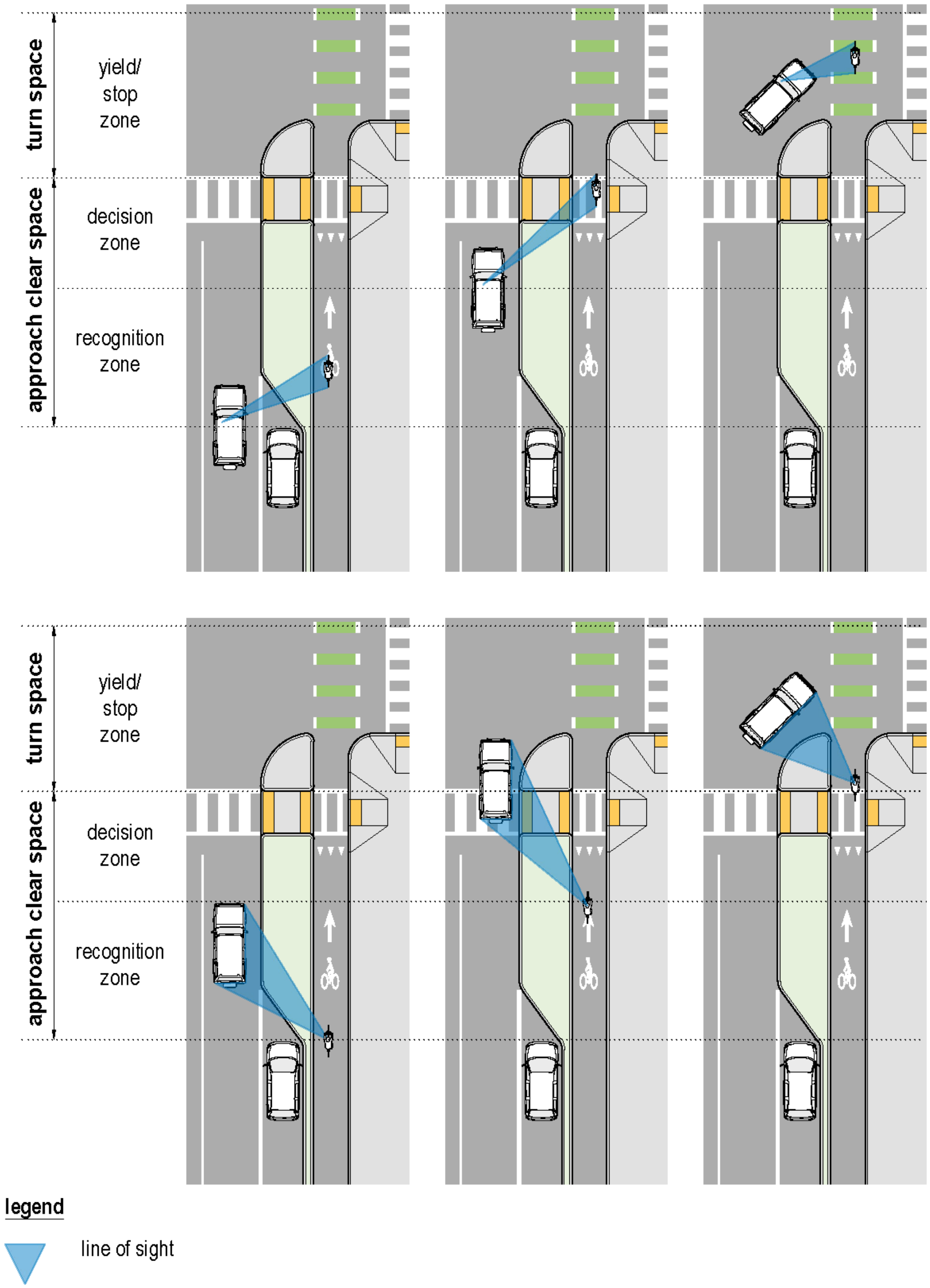


Figure 6-9. Yielding Scenarios Case A

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

Case A: Right-Turning Motorist Across Separated Bike Lane or Sidepath

Figure 6-9 depicts Case A, which applies when a motorist is making a permissive right turn across a bikeway. In this case the motorist will be decelerating for the right turn and their turning speed will be controlled by the intersection corner geometry and width of the receiving roadway. Table 6-3 identifies the minimum approach clear space, measured from the point of curvature of the motorist's effective turning radius, which represents the location where the motorist will have decelerated to the turning speed; this location may or may not be the curb line point of curvature. Providing the appropriate clear space provides the necessary sight lines between motorists and bicyclists to stop (or yield) as appropriate. For locations with two-way separated bike lanes or sidepaths, additional approach clear space is not typically required as the recognition zone between the counter-flow bicyclist movement and the right-turning motorists should exceed the recommended sight distances.

Case B: Left-Turning Motorist Across Separated Bike Lane or Sidepath

Figure 6-10 depicts Case B, which applies when a motorist is making a permissive left turn across a bikeway. On two-way streets with a two-way separated bike lane or sidepath, there are two sight lines that should be maintained. A left-turning motorist approaching a turn needs a line of sight to bicyclists approaching from the same direction. Table 6-3 identifies the minimum approach clear space based on the effective turning radius for the left-turning motorist.

The provision of Bike Case A for motorists making a right-turn across a two-way bikeway provide the necessary line of sight between a left-turning motorist and a bicyclist approaching from the opposite direction. On one-way streets with a left-side separated bike lane or sidepath, Case A should be applied for left turns as it has the same operational dynamics.

On streets with two-way traffic flow, the operational dynamic of a motorist looking for gaps in traffic creates unique challenges that cannot be resolved through improving sight distance. This is a challenging maneuver because the motorist is primarily looking for gaps in oncoming motor vehicle traffic and is less likely to scan for bicyclists approaching from behind. Unlike for Case A where the motorist is decelerating towards the crossing, the motorist in Case B will be accelerating towards the crossing once they perceive a gap in traffic. This creates a higher potential for conflicts on roads with the following:

- High traffic volumes and multiple lanes;
- Higher operating speeds; and
- High left turn volumes.

Where it is not feasible to eliminate high-speed and high-volume conflicts through signalization or other traffic control, it may be necessary to reevaluate whether turns should be permitted, whether a sidepath or two-way separated bike lane is appropriate at the location, or provide adequate space for the motorist to complete the turn and then stop or yield to potential conflicts with crossing bicyclists and pedestrians with a protected intersection configuration (see Section 6.4.5.2).

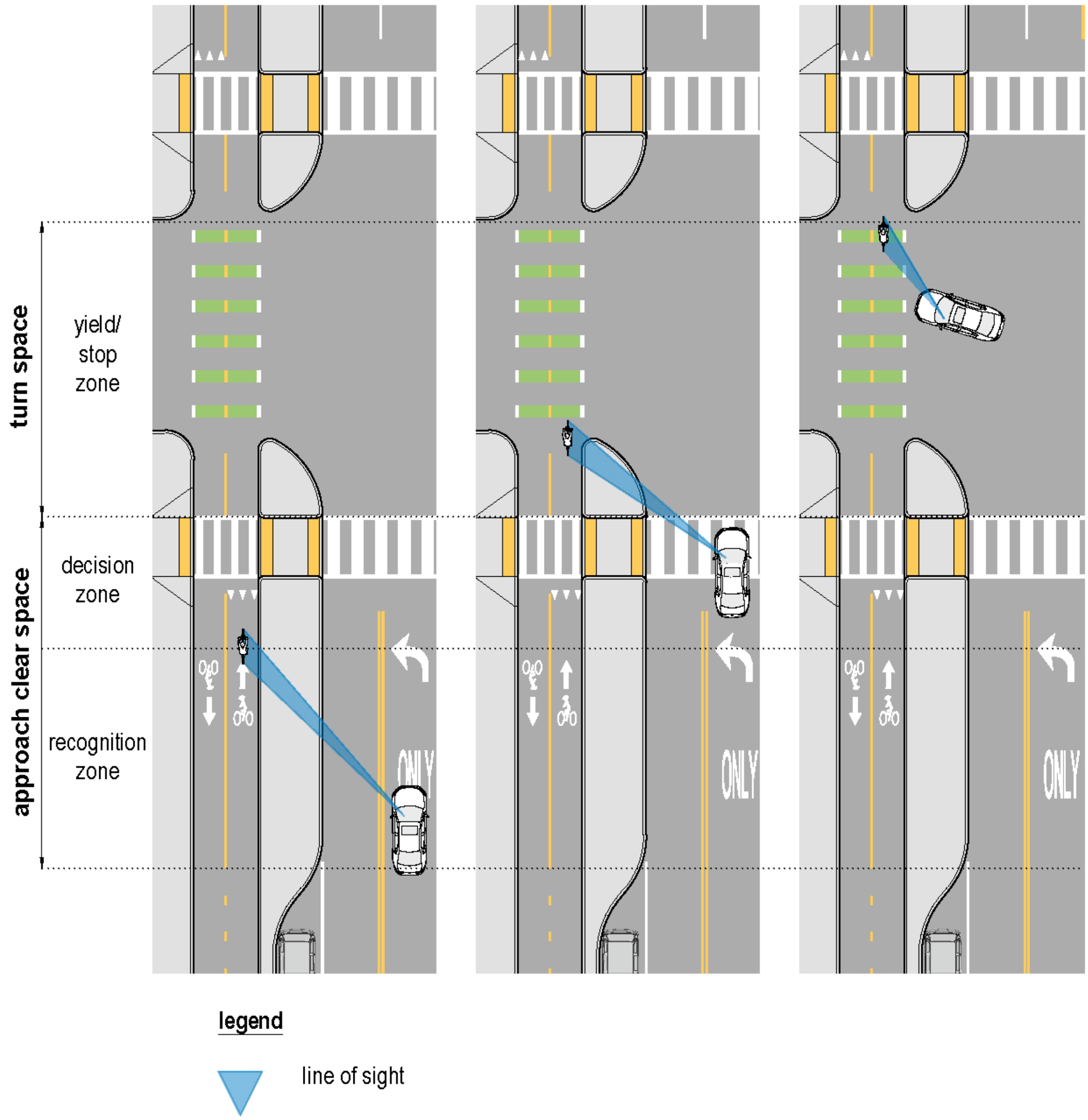


Figure 6-10. Intersection Sight Distance Case B

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.3.5 Horizontal Shifting Tapers

Changing the horizontal alignment of bike lanes and separated bike lanes may be accomplished with either horizontal curves or shifting tapers. Large radius curves which meet or exceed the guidance for horizontal curves of shared use paths may be used but will typically only be practical for use on shared use paths. In most applications within a roadway corridor, tapers will be easier to establish and are sufficient. Tapers should generally occur gradually, with a minimum length as calculated using the formula in Table 6-4.

Table 6-4. Shifting Taper Equation

Shifting Taper Equation		
$L = \frac{WS^2}{60}$		
Where:		
L	=	longitudinal lane shift (ft), minimum 20 ft
W	=	lateral width of offset (ft)
S	=	target bicyclist operating speed (mph)

Solid white lane lines should be used to guide bicyclists around obstructions at a taper of Length (L) = (1+ W (Width of Obstruction)) * Bicyclist Speed (S) along the roadway (e.g. drainage grates) or shared use paths (e.g. bollards) following the **TMUTCD Section 9C.06 and Figure 9C-8.** See guidance on design speed for each bikeway type throughout Section 4.4.

6.4.3.6 Surface Treatments

6.4.3.6.1 Shared Use Path Considerations

Paved Surfaces

It is important to construct and maintain a smooth ridable surface clear of defects, joints, and other potential obstructions on bicycle facilities. Shared use paths must also meet pedestrian accessibility surface requirements. Reinforced concrete is typically preferred for all bikeways compared with asphalt, crushed aggregate, sand, clay, or stabilized earth. Since unpaved surfaces provide less traction, they decrease braking ability for bicyclists which can cause bicyclists to lose control more easily.

Permeable Pavements

If permeable pavements are smooth, stable, and slip resistant, they can be used for bikeways. By facilitating gradual absorption of water into the ground, permeable pavement can increase bike traction and reduce icing by providing an outlet for standing water, provided that the surface is maintained to ensure continued permeability.

Locating utilities within or close to permeable pavements should be avoided to the maximum extent possible or limited to crossings of the bikeway. When repairs are necessary, utility companies sometimes do not recognize permeable pavements, and replace them with standard asphalt, reducing stormwater treatment capacity. Where crossings are unavoidable, a carrier pipe should be provided to allow access to the utility without damaging the permeable pavement.

Unpaved Surfaces

Unpaved surfaces may be appropriate on an interim basis but the preferred trail surface is paved. Interim unpaved surfaces could be used on rural shared use paths in relatively flat terrain, where the intended use of the path is primarily recreational. To accommodate people with disabilities, unpaved pathways should be constructed of materials that are firm and stable. Possible surfaces for unpaved paths include crushed stone, stabilized earth, and limestone screenings, depending upon local availability.

6.4.3.6.2 Utility Considerations

Addressing utility location may not be practical in retrofit situations where minimal reconstruction is anticipated. In those cases, care should be taken to ensure utilities will not present a hazard to bicyclists.

In new construction or substantial reconstruction presents opportunities to proactively address utility placement. Careful consideration of utilities within a roadway corridor can minimize potential utility conflicts with bicyclists and ensure adequate maintenance access for utility owners. The following should be considered:

- Avoid locating utility covers and large ventilation grates within bikeways to maintain a smooth bicycling surface and minimize detours during utility work. Where unavoidable, utility covers and large ventilation grates within bikeways should be:
  - smooth and flush with the bikeways surface,
  - minimizes the need for avoidance maneuvers by bicyclists, and
  - skid resistant to prevent a crash hazard.
- Locate poles and fire hydrants outside a minimum of 2-ft from bikeways
- Bikeways retrofitted within an existing roadway may require realignment of traffic signal heads and detection equipment.
- Where bikeways are collocated with overhead lines, designers should be aware of required clearances from both poles and overhead conductors, and ensure that site elements such as bridges, railings, retaining walls, signs, and bicyclists’ operating envelope remain outside of the required clearances.

6.4.3.6.3 Drainage Considerations

Bikeways should be designed to prevent water ponding, ice formation, and the collection of debris. The following should be considered:

- A cross slope of 1 percent typically provides adequate conveyance of drainage.
- Sloping bikeways in one direction instead of crowning is preferred and simplifies drainage and surface construction.
- Bikeways retrofitted within a road corridor may be located within existing slope and drainage patterns of the respective roadway to minimize changes to the roadway's drainage system.
- Drainage grates should be located outside the bikeway whenever practical (e.g. placed entirely within a gutter next to a bike lane) to minimize a reduction of the usable width of the bikeway.
- Drainage grates that extend into a bikeway may cause bicyclists to swerve increasing crash risks. Where this is unavoidable, to minimize crash risk the following should be considered:
  - Guide bicyclists around the grate with pavement markings (see Section 6.4.3.5)
  - Install bicycle friendly drainage grates which meet the following parameters:
  - Skid resistant to prevent a crash hazard,
  - Align grate openings perpendicular to the bicyclists' direction of travel (see Figure 6-11),
  - Limit the gap between the grate and frame to 0.5 inch or less, perpendicular to the path of travel,
  - Grates should remain smooth and flush with the bikeways surface

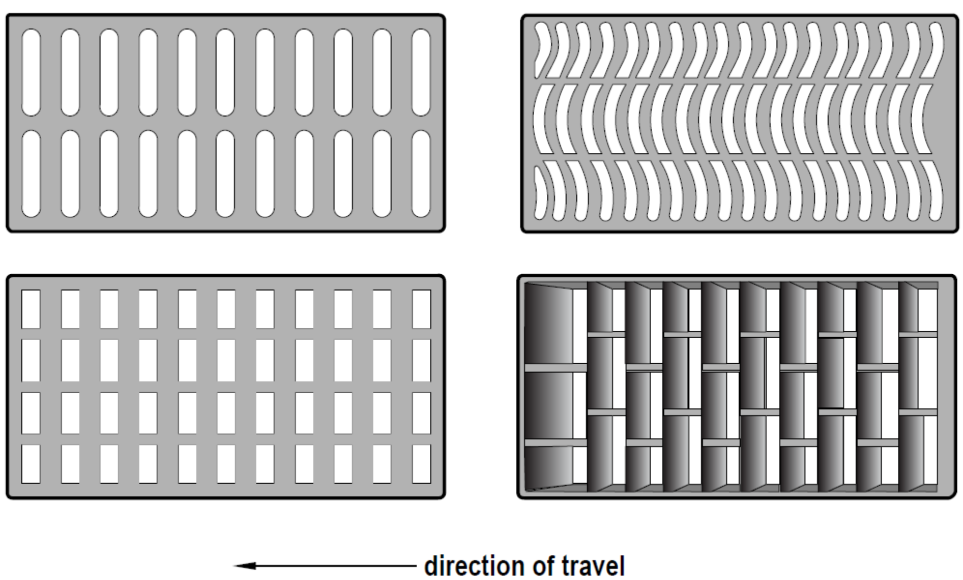


Figure 6-11. Bike Compatible Drainage Gates

6.4.3.6.4 Bikeway Curb Considerations

Some curb types can increase the risk of bicycle crashes if struck by a wheel or pedal. The face of curb angle, and curb height influence the functional width of the bikeway, crash risk to bicyclists, the ability to exit bikeways, and the risk of encroachment into the bikeway by other users. In locations where the bikeway is located between curbs, it is preferable to provide sloping curbs or reduced height curbs (less than 3-in). For curbs less than 3 inches in height there is no minimum shy space; however, shy space to other appurtenances should be carefully considered. For curbs between 3 inches and less than 6 inches in height, a desirable shy space of 1-ft, with no minimum in constrained conditions, applies. Refer to the current TxDOT Curb Standard (CCCG) for illustrations on the various TxDOT standard curb types.

Curbs with integral gutters include a longitudinal seam parallel to bicycle travel that may deteriorate, resulting in dips or ridges that increase crash risk for bicyclists. Gutters also may have uneven surfaces where street resurfacing activities do not adequately remove asphalt approaching the gutter. Where curbs are provided with integral gutter, the minimum shy space to the curb is the width of the gutter.

6.4.3.7 Railings and Barriers Adjacent to Bikeways

A barrier that separates vehicular traffic from a sidepath or separated bike lane must:

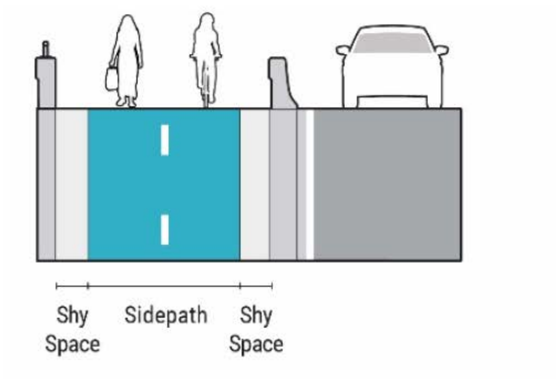
- be a minimum of 31-inches in height;
- meet the current barrier standards; and
- provide an end treatment consistent with current Roadway Design standard practice.

Where barriers are used immediately adjacent to a sidepath with minimum width dimensions, there may be risk a bicyclist could fall over the top; in these circumstances a minimum barrier height of 42-in is recommended. Barriers across bridge structures must meet the respective Bridge Railing Manual Requirements.

Barriers or railings located on the outside of a sidepath used to protect bicyclists and path users from falling into a hazard must be a minimum of 42-inches in height. A higher 48-in to 54-in continuous barrier or railing may be considered in locations where:

- bicyclist speeds are likely to be high (such as on a downgrade);
- high winds are typical (such as on bridges); or
- a bicyclist could impact a railing at a 25-degree angle or greater (such as on a curve).

Shy spaces to railings and barriers shall not be included within the width of the SUP (see Figure 6-12).



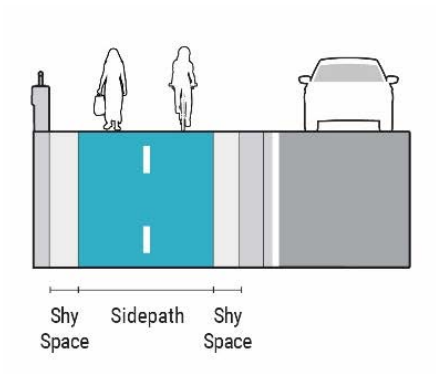


Figure 6-12. Example Bridge with Barriers for Sidepaths; High-speed (left) and Low-speed (right)

Where shared use sidepaths (SUP) or separated bike lanes (SBL) with sidewalks (SW) are provided on a bridge where the roadway cross section must narrow, the narrowing should be prioritized in the following order:

- Minimize shy distances to constrained conditions to vertical barriers (1-ft);
- Minimize street buffer to minimum width (4-ft);
- Narrow SUP width from desired to constrained minimum; or
  - Narrow SBL width from desired to minimum
  - Narrow SW width to the minimum

6.4.3.8 Intersection Elements

6.4.3.8.1 Bicycle Crossing / Conflict Markings

Where a bike lane crosses an intersection separate from a crosswalk, bike lane markings may be extended through the intersection to delineate the bicycle crossing. Bike lane crossings are desirable to:

- Delineate a preferred path for people bicycling through the intersection, especially a crossing of a wide or complex intersection;
- Improve the legibility of the bike crossing to roadway users; and
- Encourage motorist stop or yielding behavior, where motorists must merge or turn across the path of a bicyclist or pedestrian.

Figure 6-13 provides design details for various bike crossing scenarios.

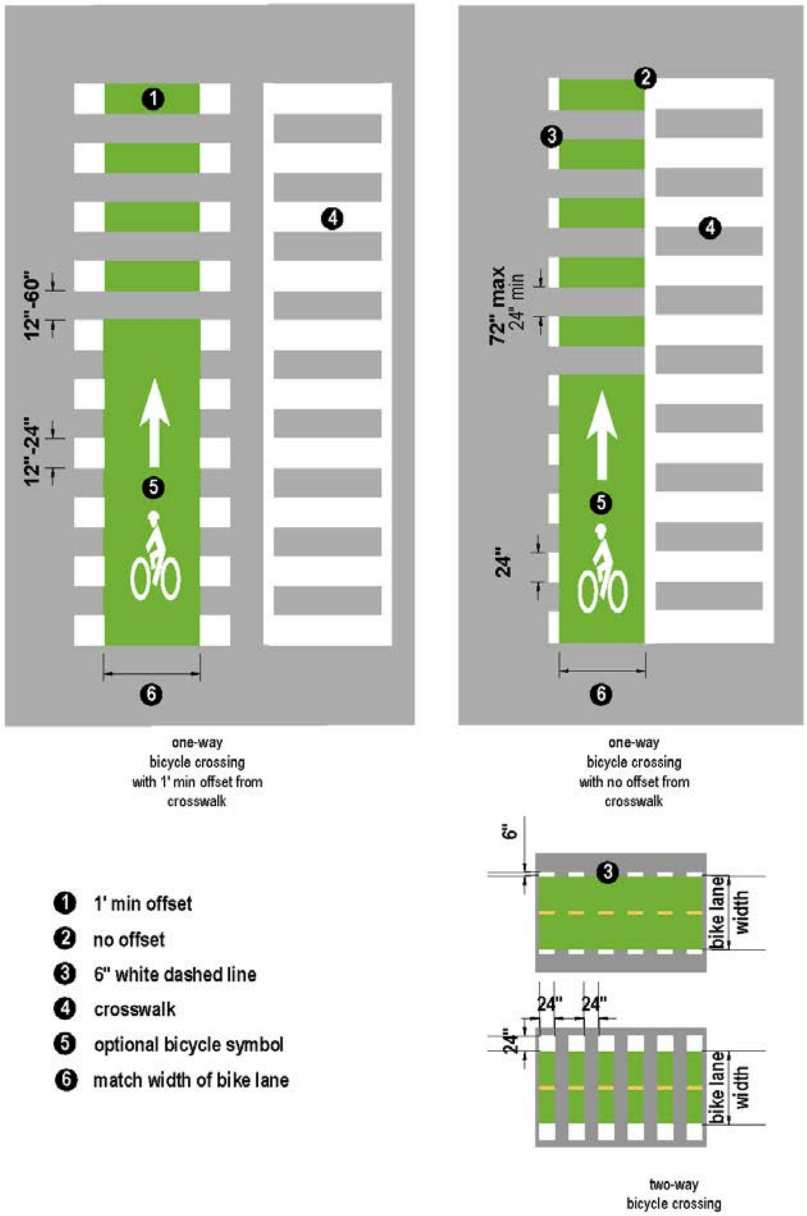


Figure 6-13. Bicycle Crossing Pavement Markings

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.3.8.2 Bicycle Signals

At signalized intersections, bicyclists may be controlled by motor vehicle signals, pedestrian signals, or bicycle signals. A bicycle signal provides a separate indication for the exclusive use of bicyclists. The use of a bicycle signal requires a "[BICYCLE] SIGNAL" (R10-10b) sign installed immediately adjacent to the signal. Bicycle signals are typically used in the following situations:

- Two-way separated bike lanes;

- One-way separated bike lanes where the right-turn or left-turn motor vehicle movements across the separated bike lane exceed the thresholds in Figure 6-14;
- Where bicyclists' position in the bikeway does not allow them to see motor vehicle or pedestrian signals that may otherwise be able to control their movement; and
- Where intersection complexity is such that signals are helpful, as determined by engineering judgment.

Bicycle signals can use a standard vehicle signal head with the R10-10b sign or may use a signal with bicycle signal faces that comply with the requirements in FHWA's Interim Approval (IA)-16. However, IA-16 has a specific requirement that bicycle signal faces may only be used if all bicycle movements are protected. An approved Request to Experiment from FHWA is needed for locations where permissive turns are allowed across a bike lane controlled by bicycle signal faces. See the TMUTCD for additional guidance on the use of Bicycle Signal Faces.

6.4.3.8.3 Considerations for Phase Separation

Designers should assess the number of right or left-turning motorists across sidepaths, raised bike lanes, or separated bike lanes which are physically separated from adjacent motorists during the peak hour to identify if permissive turns should be considered to manage potential conflicts which can arise due to their physical separation at the intersection. Figure 6-14 identifies the thresholds where signal phase separation between bicyclists and turning motorists is recommended, but other factors such as the intersection complexity, number of signal phases, and engineering judgement should guide the designer's decisions.

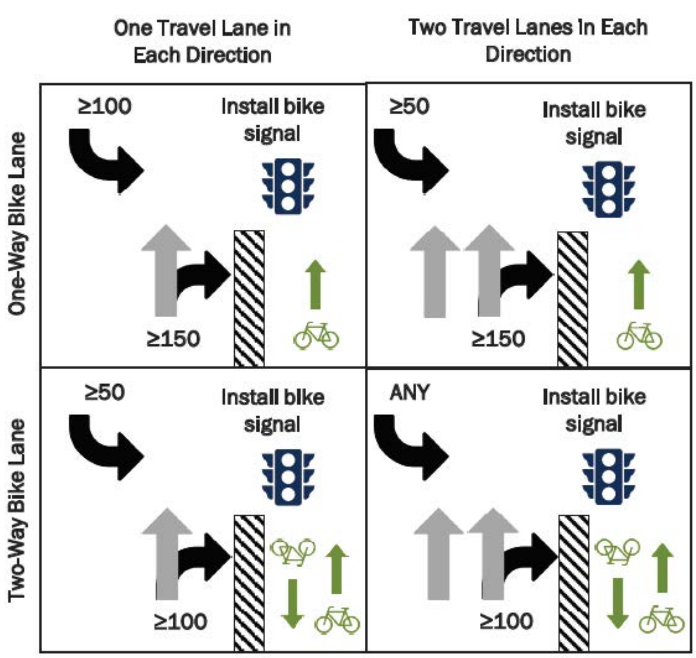


Figure 6-14. Signal Phase Separation of Turning Motorists

Bikeway Lighting

Fixed-source lighting can improve visibility along bikeways at night or under other dark conditions. Lighting can also greatly improve bicyclists' ability to detect surface irregularities under such conditions, even when their bicycles are properly equipped with headlamps. Most bicycle trips occur during daylight hours, yet a relatively high incidence of crashes occur at night and dusk/dawn on roadways without bikeways. Nationally, 48 percent of bicyclist fatalities and 28 percent of bicyclist injuries occurred during the 12-hour period between 6 p.m. and 6 a.m.

Provision of lighting should be considered where night-time use of bikeways is anticipated and especially on:

- Bikeways provide convenient connections to transit stops and stations, schools, universities, shopping, and employment areas,
- Under vehicular bridges, underpasses, tunnels, or locations with limited visibility,
- Along bridges used by bicycles and pedestrians,
- Along high use portions of bikeways that lead to areas with frequent evening events,
- At trail intersections with higher volume roadways, and
- At major trail entrances

Designers should refer to Chapter 7 Section 3.9 for information and guidance on the design of pedestrian scale lighting and appropriate illumination levels. Bikeway lighting should follow the same principles.

6.4.3.9 Restrict Motor Vehicle Use of Bicycle Facilities

Unauthorized use of shared use paths, on-street bicycle lanes, and separated bicycle lanes by motorists occurs occasionally. In general, this is a greater issue on shared use paths that extend through independent rights-of-way that are not visible from adjacent roads and properties. Per the TMUTCD, the NO MOTOR VEHICLES (R5-3) sign can be used to clarify vehicle restrictions.

The routine use of bollards and other similar barriers (e.g. z-gates, fences) placed within the clear width of a bicyclists operating path to restrict motor vehicle traffic is not recommended. Bollards should not be used unless there is a documented history of unauthorized intrusion by motorists that results in injury. Barriers such as bollards, fences, or other similar devices used to limit access to bikeways create permanent obstacles to bicyclists, which can cause serious injury. Bicyclists approaching these obstructions may shield them from a following bicyclist's view until a point where the trailing rider does not have sufficient time to react which can contribute to crashes.

Where a device to restrict motorists is determined to be necessary, consider the use of flexible or spring-mounted delineators before installing rigid bollards.

6.4.4 Bikeway Types

This section provides a description and brief design guidance for the most common bicycle facility types. From left to right, Figure 6-15 shows decreasing separation between bicyclists and motor vehicles. The bikeways presented on the left have the greatest appeal to the Interested, but Concerned bicyclist (see Section 6.4.2.3) since they have the most separation from motor vehicles.

A detailed description of each bikeway for urbanized contexts can be found starting in Section 6.4.4.2 through Section 6.4.4.8; rural contexts can be found in Section 6.4.4.9. For additional guidance not stated herein, see additional design resources listed in Section 6.4.7, which are sourced throughout this guide. It should be noted that different facility types may be used within different land use contexts or within the same land use context. Additionally, due to possible project site limitations, different facility types may be used to accommodate the origin and destination routes of bicyclists and pedestrians on each side of the roadway. For Roadways on TxDOT's Texas Bicycle Tourism Trails Example Network, provide appropriate Bike facility in coordination with the local community.

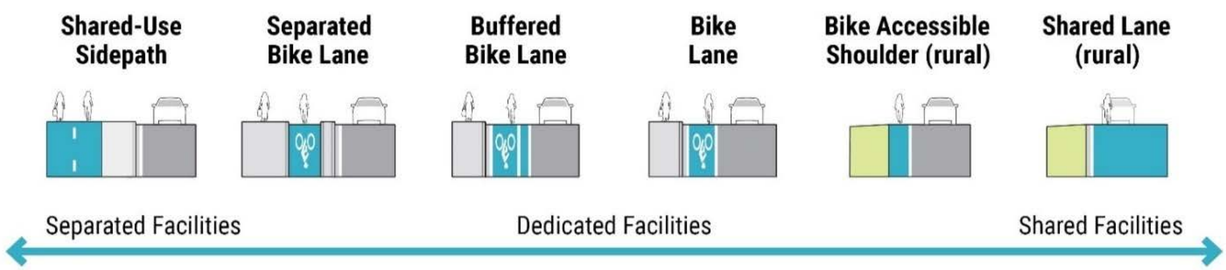


Figure 6-15. Bicycle Facility Types by Level of Separation

Chapter 9 of the TMUTCD provides detailed guidance on traffic control devices for bicycle facilities. Designers should refer to that chapter in addition to any bikeway specific guidance provided in the sections below.

6.4.4.1 Bicycling Operating Space and Bikeway Widths

The physical space for each bicycle is determined by the width of the widest portion of the bicycle which is typically the handlebars or the wheelbase on adult tricycles, child trailers, or adult box bicycles. To accommodate most bicyclists, it is recommended that the target design user be an adult bicyclist operating with a trailer to establish the following minimum physical space dimensions:

- 30-inches in horizontal width;
- 10-ft in length; and
- 7-ft in height.

Figure 6-16 shows the operating space for the bicyclist described above. The minimum operating space for a bicyclist is 42-in. To provide additional shy space to other users or vertical objects, a minimum of 6-in of additional shy space is required. For this reason, operating space is generally assumed to be 48-in for a single direction of bicycle travel, and essentially functions as a "lane" of travel for a bicyclist. This lane does not need to be marked on a bikeway unless it operationally beneficial to do so. The bikeway width minimums in the guidance below accounts for the operating and shy spaces required.

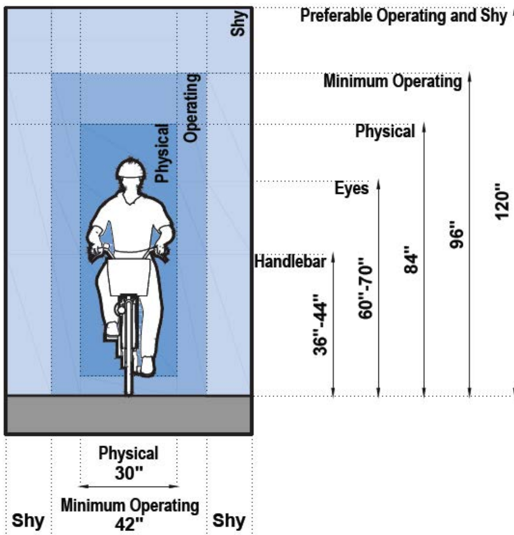


Figure 6-16. Operating Space for Bicyclists

6.4.4.2 Shared Use Paths Adjacent to Roadways (Sidepaths)

6.4.4.2.1 Description

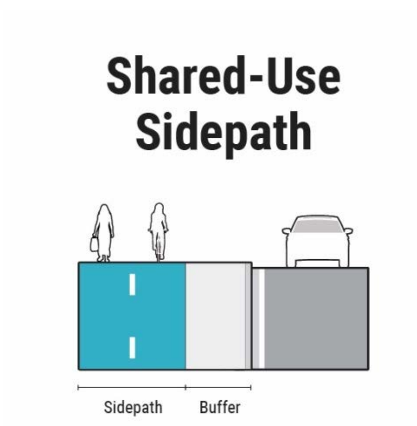


Figure 6-17. Example Shared Use Path Schematic

Shared Use Paths adjacent to roadways are located within a roadway corridor following the roadway alignment (hence the term sidepath) and are physically separated from motorized vehicular traffic by a landscaped buffer or a barrier. Sidepaths are generally designed for two-way travel, because in addition to bicyclists, users may include inline skaters, skateboarders, and pedestrians.

Sidepaths which do not provide adequate width may increase crash risk between people on the path.

Conflict points occur between motorists and path users at intersections and driveways. Of particular challenge is the contra-flow movement of higher speed users such as bicyclists. Conflict points should therefore be minimized and mitigated to the greatest extent practicable to maximize the comfort and safety of users operating on the facility (see Section 6.4.5). The consolidation of driveways will reduce conflicts within the corridor.

Shared use paths may also be constructed with an independent alignment from a roadway which typically limits conflict points to mid-block crossings of roadways. The shared use path on an independent alignment allows the vehicular driver to address the conflicts of the bicyclists exclusively, before proceeding to the roadway intersection and then addressing the standard conflicts with other vehicular traffic. The AASHTO Bicycle Design Guide and RDM should be reviewed for additional guidance of shared use paths in independent alignments. The following guidance is specific to sidepaths.

6.4.4.2.2 Basic Design Guidelines

Application

Shared use paths are particularly useful when roadway width is limited and providing an on-street bikeway is not feasible. However, before defaulting to a shared use path, careful consideration should be given to potential conflicts between pedestrians and bicyclists as it is preferable to use separated bike lanes with sidewalks in urban areas and other locations where pedestrian volume is likely to result in conflicts between bicyclists and pedestrians as discussed in the width guidance.

For roadways with high driveway density, frequent street crossings, or overall high driveway volumes a one-way bicycle facility may be a more appropriate option, with an appropriate facility provided on each side of the roadway to provide needed origin and destination points. One-way bicycle facility preferred options include a separated or buffered bike lane. As identified in the AASHTO Bicycle Design Guide, for longer distances on urban and suburban streets with a considerable number of driveways and street crossings, two-way sidepaths can create operational concerns. Additional consideration should also be given with respect to parking configurations in the driveway vicinity thar may tend to obstruct the sidepath usage or sight lines.

Width

When selecting a sidepath width, designers should consider the anticipated user volumes and mode split (bicycles, pedestrians, runners, etc.). The FHWA Shared Use Path Level of Service – Users Guide (SUPLOS) is a helpful tool to inform a decision regarding the width of a path (link to the calculator tool: https://www.fhwa.dot.gov/publications/research/safety/pedbike/05138/trail_los_calculator.cfm).

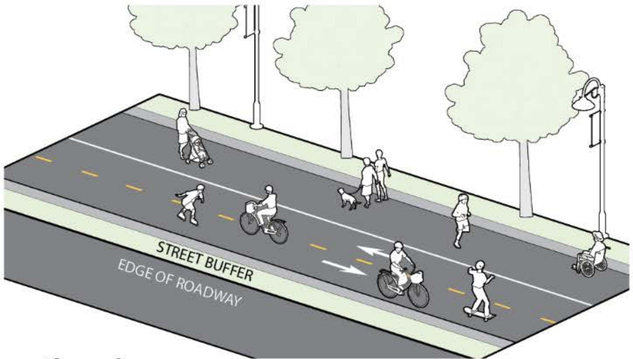
The inputs into the SUPLOS calculator include the path width, trail user volume (one direction), trail user mix (mode split), and presence of a centerline stripe; the output is the calculated SUPLOS. A SUPLOS of “C” or better is desirable over the life of the facility to ensure it is comfortable and safe for all users. Table 6-5 provides an example calculation and set of typical inputs.

As a starting point for a new facility (without a shared use path), bike and pedestrian counts should be taken during the anticipated peak hour period on the existing roadway, and/or an analogous parallel facilities which may provide a component of latent demand for the new shared use path. A reasonable factor should be applied for anticipated future growth of share use path usage. The *Texas Bicycle and Pedestrian Count Exchange* (https://mobility.tamu.edu/bikepeddata/) contains pedestrian and bicyclist count data for various facilities statewide and can be used as an additional source of information.

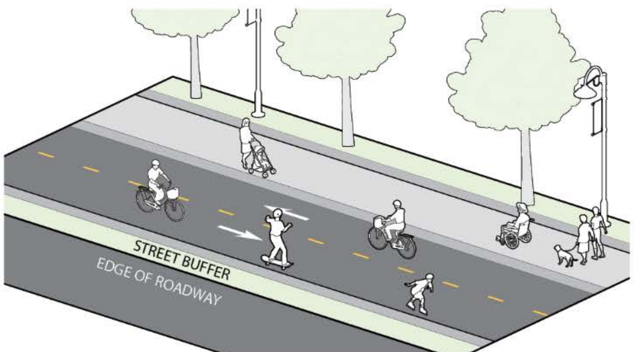
The desired width for a sidepaths may range from 11-ft to 15-ft or more, depending on the SUPLOS calculation. These widths will accommodate higher user volumes while minimizing conflicts between them (based on the SUPLOS output). On a 12-ft sidepath, three “lanes” of users can operate simultaneously, allowing people to operate side-by-side while being passed by a person in the other direction (see Section 6.4.4.1). To maximize service life and to assure a reasonable SUPLOS grade, paved widths should not be less than 10-ft.

As path user volumes increase, designers should consider increasing the width of the sidepath up to 15-ft. As path widths begin to exceed 15-ft in width, it may be desirable to separate pedestrians from bicyclists into sidewalk and separated bike lanes to minimize speed differential between pedestrians and wheeled users.Where it is determined to separate bicyclists and other higher-speed wheeled users from pedestrians (including people walking or using wheelchairs or other assistive devices) there are four strategies which can be considered (see Figure 6-18):

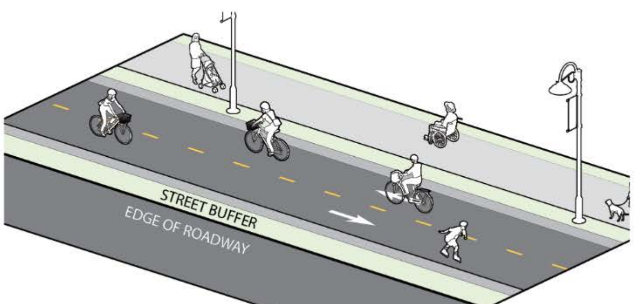
1. A wide path can be provided which separates users using pavement markings;
2. A wide path can be provided which separates users with a traversable surface delineation; or
3. Separate paths can be used which are physical separated from each other.
4. A separate bicycle facility type may be selected.



option 1



option 2



option 3

Figure 6-18. Potential Options for Separation of Shared Use Paths

The standard **minimum width** is 10-ft. A **minimum width** of 8-ft may be used in rare circumstances where all the following conditions are met:

- Bicycle traffic is expected to be low (< 50 bicyclists/hour) during peak hours on the path;
- Pedestrian use of the facility is not expected to be more than occasional or less than 30% of total path traffic;

- Horizontal and vertical alignments provide frequent, well-designed passing and resting opportunities where the width is at least 10-ft; and
- The path will not be regularly subjected to maintenance vehicle loading conditions that would cause pavement edge damage.

In **constrained conditions**, a minimum width of 8-ft may be used for short distances to avoid physical constraints such as bridge abutments, utility structures, and environmental constraints.

Table 6-5. SUP LOS calculation for high foot traffic example

Segment Name	Path Width	Centerline	Volume (users per hour in 1 direction) and Mode Split						
Name	**Width (ft)**	**1=Centerline**	**One-Way (per hour)**	**Adult Bicyclists**	**Peds**	**Runners**	**In-Line Skaters**	**Child Bicyclists**	**SUPLOS grade**
More Peds	12.0	0	100.0	20.0%	60.0%	15.0%	2.0%	3.0%	C

Street Buffer Width and Considerations

The space between the inside edge of sidepath and adjacent edge of outside travel lane (uncurbed), or face-of-curb (curbed) is the street buffer. It is desirable to provide as much distance from the road as is practicable given project conditions.

The **minimum street buffer** is shown in Table 6-6.

Table 6-6. Two-Way Sidepath Minimum Street Buffer Widths

Roadway and speed condition	Minimum street buffer
Curbed low-speed (45 mph or less)	4-ft from FOC [1]
Curbed high-speed (50 mph or greater)	6-ft from FOC [1]
Uncurbed all speeds	10-ft

Note:

1. For curb and gutter sections with a shoulder, bike lane or any buffer in addition to the curb offset (excluding turn lanes, accel lanes, or decel lanes), the minimum street buffer measurement begins at the edge of through travel lane. This minimum width is 10-ft. The required minimum street buffer for the sidepath then becomes the lesser of the 10-ft measurement, or the face of curb (FOC) minimum in Table 6-6.

The **desirable street buffer** would be the respective desirable clear zone values from RDM Table 2-12. Increasing the buffer width in the vicinity of driveways provides better alignment and visibility and is beneficial to all users.

A minimum shy space of 2-ft should be provided to intermittent (e.g. signs, streetlights, utility poles) and continuous (e.g. walls, railings, fences, barriers) vertical objects to minimize crash risks and increase the comfort of path users. Where space is available, wider shy spaces are desirable. In constrained conditions, this shy space may be reduced to 1-ft except to post mounted signs.

A graded shoulder with a minimum 2-ft width, 5-ft desirable, with a maximum cross-slope of 1V:6H should be provided on both sides of all shared use paths where natural terrain is present adjacent to the path alignment.

Where a bikeway is adjacent to a hazardous condition such as a parallel body of water or steep downward slope, a shoulder between the bikeway and the top of the slope of 5-ft is desirable. For steep slopes where the shoulder is less than 5-ft, physical barriers or rails are recommended in Chapter 7 Section 3.12. Specific railing considerations for bicyclists are provided in Section 6.4.3.7.

Signing and Marking

Signs installed in the vicinity of sidepaths shall be installed in accordance with the applicable TMUTCD criteria. The TMUTCD requires all signs be located a minimum of 2-ft laterally from the nearest edge of a shared use path.

The MUTCD requires that no portion of a sign or its support be placed within 2-ft laterally from the near edge of a shared use path. Where space is available, wider shy spaces are desirable.

See **TMUTCD – Section 9C.03 Marking Patterns and Colors on Shared-Use Paths** for information about signing and marking for sidepaths. Additional information is provided in Section 6.4.5.3 regarding mid-block shared use path crossings.

Bicyclist Design Speed

A target design speed of 15 mph is generally appropriate to accommodate bicyclists on a shared use sidepath given the fact bicyclists are operating with pedestrians and may encounter driveways and intersections due to the alignment alongside the roadway.

For shared use paths with lower volumes of users expected now and in the future, a design speed of 18 to 30 mph may be appropriate where pedestrian volumes are low (less than 30 percent) and the primary purpose of the shared use path is to provide a higher speed bicycling opportunity between destinations.

On unpaved path surfaces, bicyclists tend to travel slower to compensate for reduced braking ability, so a lower design speed of 12 mph may be used.

Cross Slope and Grade

Cross slopes of 1 percent are more comfortable for people with disabilities and people bicycling with more than two wheels (e.g., cargo bike, adult tricycles, or trailers); to meet pedestrian accessibility guidelines the cross slope cannot exceed 2 percent.

For shared use paths in independent alignments, longitudinal grades must meet pedestrian accessibility guidelines as they are expected users of the facility. While final standards have not been adopted, the guidelines described in the U.S. Access Board's *Supplemental Notice of Proposed Rulemaking* (SNPRM) on Shared Use Paths provide the best available information on accessibility of shared use paths.

Therefore, grades on shared use paths on an independent alignment are limited to 5 percent to maximize pedestrian accessibility. The SNPRM acknowledges that certain conditions such as physical constraints (existing terrain or infrastructure, notable natural features, etc.) or regulatory constraints (endangered species, the environment, etc.) may prevent full compliance with the 5 percent maximum grade. In those cases, mitigations should be considered to assist pedestrians.

Options to mitigate excessive grades on shared use paths include the following:

- Use higher bicycle design speeds for horizontal and vertical alignments, stopping sight distance, and other geometric features;
- When steep grades occur over a long distance, consider an additional 4 to 6-ft of width to permit slower bicyclists to dismount and walk uphill, and to provide more maneuvering space for faster downhill bicyclists (see Chapter 7 Section 3.3.3);
- Install the Hill Warning Sign for Bicyclists (W7-5) and advisory speed plaque in conditions where the steep grade is unexpected and not visible;
- Exceed minimum shy distances, add recovery areas and/or protective railings;
- If other designs are not practicable, use a series of short switchbacks to traverse the grade. If this is done, an extra 4-ft to 6-ft of path width is recommended to provide maneuvering space;
- Provide resting intervals with flatter grades to permit users to stop periodically and rest; and
- Consider the provision of accessible pedestrian handrails.

When shared use paths are within the same alignment of the roadway, the sidepath grades will typically match the grade of that adjacent roadway. Where practical, a sidepath on a roadway with a grade that exceeds 5 percent should be designed to meet accessibility guidance to the maximum extent feasible.

6.4.4.2.3 Horizontal Geometric Design

Table 6-7 provides the minimum radii for horizontal curves for paved shared use paths for various design speeds. These values use a 20-degree lean angle.

In general, a greater than 60 ft radii is preferred for horizontal alignment. For Shared Use Paths 10-ft or less in width in constrained locations where environmental or physical constraints limit the ability to apply a 60-ft radius, slower design speeds (12 to 16 mph) may be appropriate to allow the use of sharper horizontal curves to minimize impacts; however, the design should not result in a path with continuous combination of sharper curve. In these situations, wider paths or curve widenings of 2-ft to 4-ft should be considered to let users navigate the effects of substandard curves.

The AASHTO *Bike Guide* should be consulted where it is desired to develop curves using the superelevation method keeping in mind that shared use paths must meet accessibility guidelines which limit cross slopes to a maximum of 2 percent.

Table 6-7. Minimum Radii for Horizontal Curve on Paved Shared Use Path at a 20-degree Lean Angle

Design Speed (mph)	Minimum Radius (ft)
10	18
12	27
14	36
15	41
16	47
18	60
20	74
25	115
30	166

6.4.4.2.4 Vertical Geometric Design

Table 6-9 provides the vertical curve length needed based on stopping sight distance on crest vertical curves. Table 6-8 provides the equation for calculating this length for scenarios not shown in Table 6-9. A recumbent bicyclist is recommended to establish design criteria and the following assumptions should be used:

- The eye height of the recumbent bicyclists is 3.83-ft; and
- The object height is at the ground surface (i.e. 0-in) to recognize that impediments to bicycle travel can exist at pavement level.

Table 6-8. Crest Vertical Curve Equation.

Length of Crest Vertical Curve to Provide Sight Equations

when $S > L$ $\quad L = 2S - \dfrac{200(\sqrt{h_1} + \sqrt{h_2})^2}{A}$

when $S < L$ $\quad L = \dfrac{AS^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2}$

where:

L	=	minimum length of vertical curve (ft)
A	=	algebraic grade difference (percent)
S	=	stopping sight distance (ft)
h_1	=	eye height (3.83 ft for a typical recumbent bicyclist)
h_2	=	object height (0 ft)

Table 6-9. Minimum Vertical Curve Length Based on Stopping Sight Distance

Minimum Length of Crest Vertical Curve (ft) Based on Stopping Sight Distance														
A	S = Stopping Sight Distance (ft)													
(%)	40	60	80	100	120	140	160	180	200	220	240	260	280	300
2									17	57	97	137	177	217
3						25	65	105	145	185	225	265	307	352
4				9	49	89	129	169	209	253	301	353	409	470
5			7	47	87	127	167	211	261	316	376	441	512	587
6			32	72	112	154	201	254	313	379	451	530	614	705
7		11	51	91	132	179	234	296	366	442	526	618	716	822
8		24	64	104	150	205	267	338	418	505	602	706	819	940
9		35	75	117	169	230	301	381	470	569	677	794	921	1057
10	3	43	84	131	188	256	334	423	522	632	752	883	1023	1175
11	10	50	92	144	207	281	368	465	574	695	827	971	1126	1292
12	16	56	100	157	226	307	401	508	627	758	902	1059	1228	1410
13	21	61	109	170	244	333	434	550	679	821	978	1147	1331	1527
14	25	66	117	183	263	358	468	592	731	885	1053	1236	1433	1645
15	29	70	125	196	282	384	501	634	783	948	1128	1324	1535	1762
16	32	75	134	209	301	409	535	677	836	1011	1203	1412	1638	1880
17	35	80	142	222	320	435	568	719	888	1074	1278	1500	1740	1997
18	37	85	150	235	338	461	602	761	940	1137	1354	1589	1842	2115
19	40	89	159	248	357	486	635	804	992	1201	1429	1677	1945	2232
20	42	94	167	261	376	512	668	846	1044	1264	1504	1765	2047	2350
Shaded area represents S=L. Minimum length of vertical curve = 5 ft														

6.4.4.2.5 Other Considerations

Crashworthy barriers, when used on locations with sidepaths, should be located between the roadway and sidepath. A minimum of 1-ft of shy distance is needed between the vehicular traffic and the barrier, and between the sidepath traffic and the barrier.

On roadways which do not have roadside obstacles requiring a barrier, a barrier or railing may be considered to improve path user safety where the facility has all these conditions present:

- The road is uncurbed;
- If shoulders are present, the road has shoulders less than 4-ft in width; and
- The road is a high-speed facility.

Section 6.4.3.7 provides additional details on the design of barriers and railings. Designers should refer to Chapter 7 Sections 3.7.3.1 and 3.7.3.2 in the Pedestrian chapter for further details on bridges and underpasses for shared use paths.

To improve the safety of contra-flow bicyclists who may be unexpected by motorists, the following measures should be considered during design:

- Provision of 6 to 20-ft street buffers at crossings to create space for motorists to stop and yield to path users while turning;
- Provision of clear sight lines allowing motorist to see approaching bicyclists, pedestrians, and motorists;
- Provision of traffic control signs, and clearly marked bicycle crossings following TMUTCD (Part 9) to alert motorists of bicycle travel;
- Reduction and consolidation of driveways to the greatest extent practicable to reduce conflict points;
- Constructing driveways at sidewalk level to emphasize bicycle use along a sidepath to motorists entering/exiting driveways to slow turning motorists and improve their likelihood of stopping and yielding to crossing path users; and
- At signalized intersections consider:
  - Provide a leading interval or phase separation to minimize conflicts where path volume exceeds 100-150 users/hour and right turning vehicle volume exceeds 100 vehicles/hour;
  - Provide a leading interval or phase separation to minimize conflicts where path volume exceeds 100-150 users/hour and left turning vehicle volume exceeds 50 vehicles/hour or roadway operating speeds exceed 35 mph; and
  - Prohibit right turns on red where vehicles frequently block the crosswalk.

Each end of a sidepath should directly connect to an on-street bike facility, another trail or path, or to a bicycle-compatible local street. Where no interconnecting bikeways exist, advanced signage should be installed informing roadway users that the path ends, and bikes may use the full lane. Additional signage in conformity with TMUTCD can be provided that directs bicyclists to interim facilities along alternate routes.

The minimum vertical clearance to obstructions over the path that may be used in constrained areas is 8-ft. The desirable vertical clearance to obstructions is 10-ft. In some situations, vertical clearance greater than 10-ft may be needed to permit passage of maintenance and emergency vehicles, or where equestrian use may be expected.

6.4.4.3 Separated Bike Lanes

6.4.4.3.1 Description

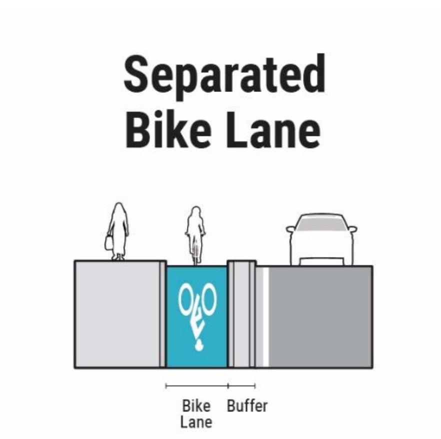
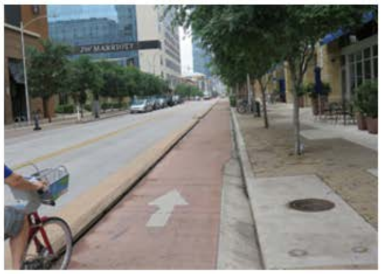


Figure 6-19. Example Separated Bike Lane Schematic

A separated bike lane is a bicycle lane that is physically separated from the adjacent motor vehicle traffic by vertical elements in the street buffer. They typically are designed to operate one-way but may also operate two-way. These are sometimes also referred to as protected bike lanes. Separated bike lanes combine the user experience of a shared use sidepath with a designated area for bike use only like a conventional bicycle lane, separate from pedestrians. They are distinct from the sidewalk but may be at sidewalk level (see Cambridge, MA, example). Vertical elements separating the bike lane from the travel lane may include continuous raised medians, flexible posts, intermittent concrete curbing (see Austin, TX, example), or parked vehicles.

Separated bicycle lanes are more appealing to a wider range of bicyclists on higher volume and higher speed roads than striped bike lanes. They avoid the conflict with an opening car door and prevent motor vehicles from driving, stopping, or waiting in the bikeway. They also provide greater comfort to pedestrians by separating them from bicyclists operating at higher speeds and further separating pedestrians from motor vehicles. Full guidance can be found in FHWA's Separated Bike Lane Planning and Design Guide and the Massachusetts DOT Separated Bike Lane Planning & Design Guide.



Austin, TX.
Street Level with Concrete Curb Separation

Cambridge, MA.
Sidewalk Level with Parking Separation

Raleigh, NC.
Street Level with Flexible Post Separation

Figure 6-20. Examples of Separated Bike Lanes by Separation Types

The cross section of a separated bike lane has three distinct zones (see Figure 6-21):

- Bike lane – The bike lane is the space in which the bicyclist operates. It is located between the street buffer and the sidewalk buffer.
- Street buffer – The street buffer separates the bike lane from motor vehicle traffic.
- Sidewalk buffer – The sidewalk buffer separates the bike lane from the sidewalk.

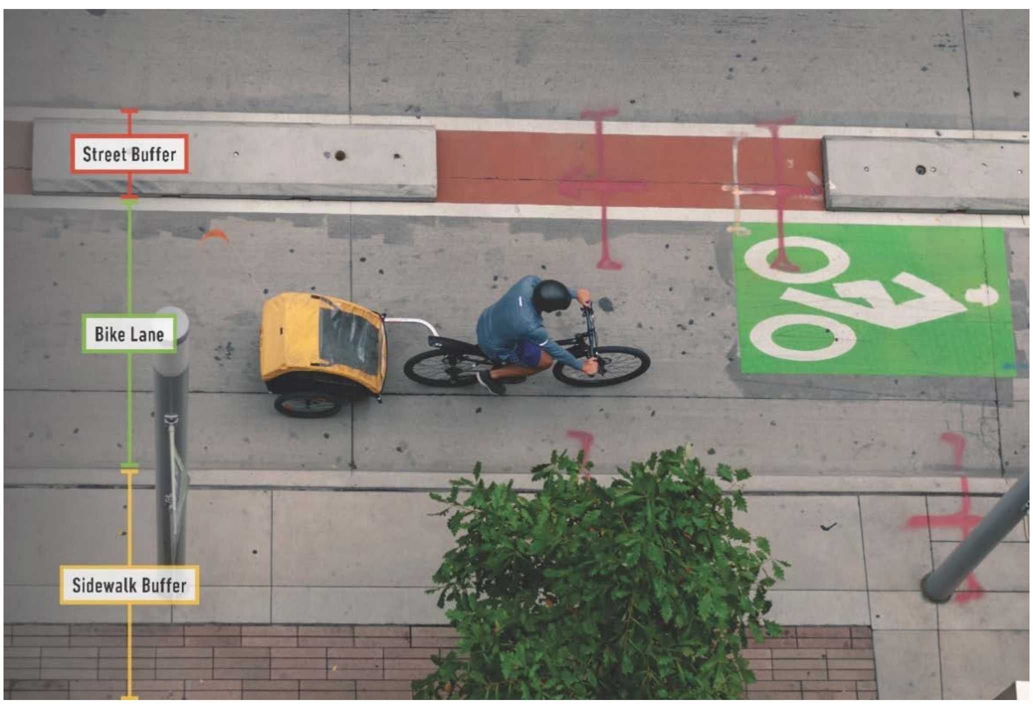


Figure 6-21. Separated Bikeway with Three Distinct Zones

6.4.4.3.2 Basic Design Guidelines

Application

Raised medians, curbs, or other low-profile, hard separated bike lanes should only be used in locations with speeds of 45 mph or less.

Separated bike lanes with flexible posts or crashworthy barriers are allowable for high-speed roadways.

Raised separated bike lanes (at sidewalk level) are allowable for all speeds.

Bike Lane Width

The desirable width of a separated bike lane depends upon the volume of users and the context of the design as shown in Table 6-10 for one-way separated bike lanes. Two-way separated bike lane widths should follow the previous guidance in Section 6.4.4.2 (Shared Use Path widths). The use of constrained dimensions should only be considered:

- for limited distances.
- as an interim measure where the larger values will result in the preferred design not being constructible.
- at locations with low volumes of bicyclists where those volumes are anticipated to remain low (< 50 bicyclists/hour)

Table 6-10. One-Way Separated Bike Lane Widths (Minimum to Desirable)

Peak Hour Directional Bicyclist Volume	Between Vertical Curbs or Flex Posts (ft)	At Sidewalk Level (ft)
< 150	6.5 – 8.5	5.5 – 7.5
150 - 750	8.5 – 10	7.5 – 9
> 750	> 10	> 9
Constrained Condition*	5	4

*Peak Hour Directional Bicyclist Volume not applicable

Buffer Widths and Considerations

The sidewalk buffer zone separates the sidewalk from the separated bike lane to communicate the sidewalk and the separated bike lane are distinct spaces. There is no minimum width, however, it is necessary to consider the needs of vegetation or street furniture if that is provided as the buffer. Sidewalks buffers need to include a detectable edge so pedestrians with vision disabilities can distinguish between the bike lane zone and the sidewalk. The sidewalk buffer zone may include:

- Street furniture or other vertical elements (such as a row of street trees),

- Continuous landscaping, or
- A curb to separate the bike lane from the adjacent sidewalk.

Separated bicycle lanes that are raised to sidewalk level with a wider buffer from traffic provide a high level of separation from traffic, but often require road reconstruction. The street buffer between the bike lane and travel lanes is needed to improve bicyclists’ comfort and to create space for vertical elements to reinforce the separation.

- For one-way separated bike lanes separated from vehicular traffic by a raised median or curb (low-speed), a minimum 2-ft buffer (measured from face of curb to face of curb) is required (see Separated Bike Lane depiction above).
- For non-curbed separated bike lanes, a minimum of a 2-ft buffer is required for low-speed conditions, and 3-ft for high-speed conditions.
- Additional buffer area is recommended to improve bicyclists’ comfort and to increase separation between bicyclists and vehicular traffic.
- See the Section 6.4.4.2 for guidance on buffers on two-way separated bicycle lanes. Two-way separated bicycle lanes also need to meet the guidance in Section 6.4.4.2 and Section 6.4.5 with respect to intersecting driveways, and conflict points.
- In scenarios where there is none, or limited space for a street buffer, designers may consider a raised bike lane, see Section 6.4.4.6.

Continuous or intermittent vertical elements such as raised medians or flexible delineator posts are needed in the street buffer to provide separation between motor vehicle traffic and the bike lane. See section 6.4.4.4 for additional guidance for the marking of buffers.

Signing and Marking

While the TMUTCD does not provide specific guidance for marking separated bike lanes, these bikeways can be marked following the best practices for shared use paths and bike lanes following the guidance in this document and the TMUTCD.

Bicyclist Design Speed

For separated bike lanes in urban areas, a target design speed of 15 mph is generally appropriate. Higher design speeds may introduce safety challenges where bicyclists interact with motorists and pedestrians. Lower bicyclist and motorist operating speeds at conflict points allow bicyclists and motorists more time to perceive potential conflicts. This is particularly important at intersections with separated bike lanes.

Cross Slope and Grade

Cross slopes of 1 percent are more comfortable for people with disabilities and people bicycling with more than two wheels (e.g., cargo bike, adult tricycles, or trailers); however, cross slope may match existing roadway conditions without a maximum where necessary.

Other Considerations

Separated bicycle lanes that are protected from traffic by a row of on-street parking offer a greater degree of separation. Additional vertical elements may be required to keep parked vehicles within the parking lane.

During the concept design stage for a separated bikeway, it is necessary to determine if it would be more appropriate to place a one-way separated bike lane on each side of the street, or to place a two-way separated bike lane or sidepath on one or both sides of the street. Selecting the appropriate configuration requires an assessment of many factors, including safety, overall connectivity, ease of access, available right-of-way, curbside uses, intersection operations, ingress and egress at the termini, maintenance, and feasibility. Table 6-11 and Table 6-12 provide an overview of trade-offs for various scenarios.

Table 6-11. Separated Bike Lane Configurations on a One-Way Street

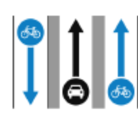
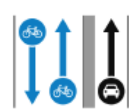
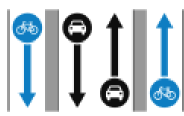
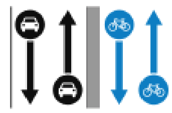
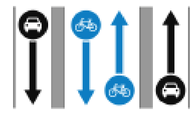
Corridor-level Planning Considerations	One-way SBL	Counterflow SBL	One-way SBL Plus Counterflow SBL	Two-way SBL
				
Access to Destinations	Limited access to other side of street		Full access to both sides of street	Limited access to other side of street
Network Connectivity	Does not address demand for counterflow bicycling, may result in wrong way riding	Requires bicyclists traveling in the direction of traffic to share the lane (may result in wrong way riding in the SBL); counterflow progression through signals may be less efficient	Accommodates two-way bicycle travel, but counterflow progression through signals may be less efficient	
Crash Risk	Lower because pedestrians and turning drivers expect concurrent bicycle traffic	Higher because pedestrians and turning drivers may not expect counterflow bicycle traffic		
Intersection Operations	May use existing signal phases; bike phase may be required depending on volumes	Typically requires additional signal equipment; bike phase may be required depending on volumes		

Table 6-12. Separated Bike Lane Configurations on a Two-Way Street

Corridor-level Planning Considerations	One-way SBL Pair	Two-way SBL	Median Two-way SBL
			
Access to Destinations	Full access to both sides of street	Limited access to other side of street	Limited access to both sides of street
Network Connectivity	Accommodates two-way bicycle travel		
Crash Risk	Lower because pedestrians and turning drivers expect concurrent bicycle traffic	Higher because pedestrians and turning drivers may not expect counterflow bicycle traffic	Higher because pedestrians and turning drivers may not expect counterflow bicycle traffic, but median location may improve visibility and create opportunities to separate conflicts
Intersection Operations	May use existing signal phases; bike phase may be required depending on volumes	Typically requires additional signal equipment; bike phase may be required depending on volumes	

NOTE: A raised curb separator on a separated bike lane may be a tripping hazard if it is located near a pedestrian path. Mitigation may include high visibility markings, offset the raised curb separator from the crosswalk, and add flexible delineators.

6.4.4.4 Buffered Bike Lanes

6.4.4.4.1 Description

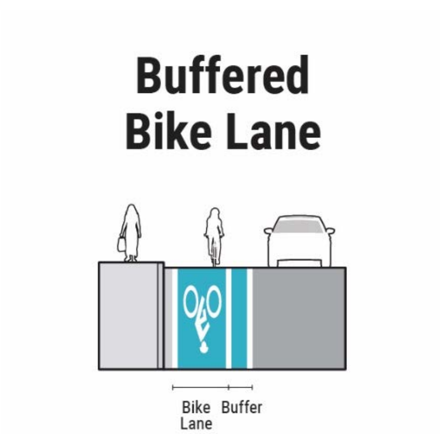


Figure 6-22. Example Buffered Bike Lane Schematic

A buffered bike lane is a one-way bike lane that is separated from the adjacent motor vehicle lane or parking lane by a striped buffer area that may include chevrons, diagonal lines, or wide pavement marking stripes. When sufficient roadway width is present, or if the number of travel lanes is reduced, a buffer may be striped between a bike lane and travel lane to provide additional comfort for both bicyclists and motorists. This provides space for bicyclists to pass one another or ride side by side without encroaching into a motor vehicle travel lane. The buffer adds to the perception of safety and encourages greater use of the on-street bicycle network. Providing added separation between motorists and bicyclists who may be traveling at substantially different speeds appeals to a wider array of bicycle users.

6.4.4.4.2 Basic Design Guidelines

Application

See Figure 6-5 in Section 6.4.2.4 for initial recommendations on when to use a buffered bike lane; the criteria below defines the criteria for all available applications for all speeds.

Bike Lane Width

The desirable useable width of a buffered bike lane is 5 to 7-ft exclusive of the buffer. The minimum useable width is 4-ft exclusive of the buffer. The usable width of the bike lane is measured from the outside buffer stripe to either the gutter joint or 1-ft from the nominal face of a monolithic curb.

Buffer Width and Considerations

Buffers should be a minimum of 2-ft wide for speeds of 45 mph or less, and 3-ft wide for 50 mph or greater.

Buffers generally consist of a combination of standard longitudinal markings and crosshatching. Diagonal or chevron crosshatch markings are optional; however, they are recommended in locations with buffers that exceed 2-ft in width.

Where provided, crosshatching should be provided at a regular interval. See Figure 6-23 for examples of bike buffer lane types.

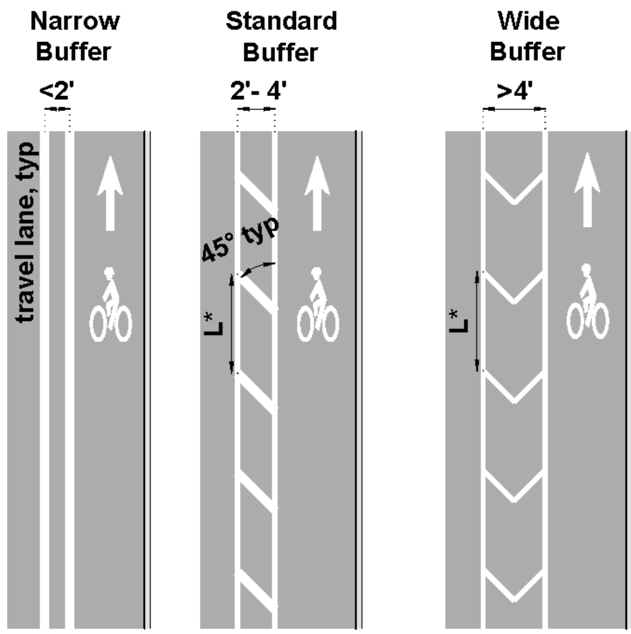


Figure 6-23. Examples of Buffered Bike Lane Types

Signing and Marking

The stripe near the travel lane should be six inches wide, while the stripe near the bicycle lane can be four inches. If buffer is 3-ft wide, diagonal hatching should also be marked. If buffer is wider than 3-ft, chevron hatching should also be marked.

Buffers can be striped between travel lanes and bike lanes, between bike lanes and parking lanes, or both.

Bicycle markings and signage should be used and are the same as a conventional bike lane.

Bicycle Design Speed

Not applicable as the roadway will be designed to accommodate motorist target design speeds which will exceed bicyclists.

Cross Slope and Grade - Not applicable.

Other Considerations - Not applicable.

6.4.4.5 Bike Lanes

6.4.4.5.1 Description

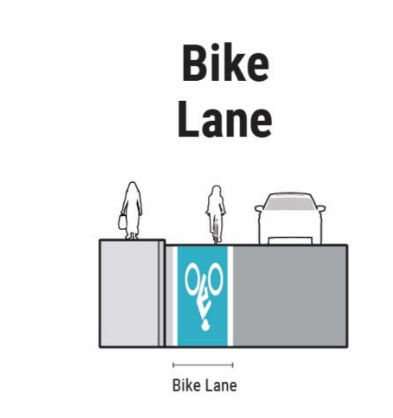


Figure 6-24. Example Bike Lane Schematic

Bike lanes are one-way facilities on a roadway that typically carry bicycle traffic in the same direction as adjacent motor vehicle traffic. Bike lanes are provided for the exclusive use of bicyclists and are identified through signage, striping, or other pavement markings. These lanes allow bicyclists to ride at comfortable speeds and encourage a position within the roadway where they are more likely to be seen by motorists. Bike lanes are typically on the right side of the street, between the outside travel lane and curb, parking lane, or road edge. While the bike lane distinguishes predictable areas for bicyclist and automobile movement, bicyclists may leave the bikeway to pass other bicyclists or avoid debris and other traffic conflicts.

6.4.4.5.2 Basic Design Guidelines

Application

Bike lanes should only be used in locations with speeds of 45 mph or less. For high-speed locations, a buffered bike lane is recommended.

In locations where space allows and additional protection and conspicuity is desired, designers may consider a raised bike lane, see Section 6.4.4.6. This can be particularly beneficial at intersections, see Section 6.4.5.

Width

The width of a bike lane has a significant impact on a bicyclists' comfort and their central operating position within a bike lane. See Table 6-13 for one-way standard bike lane width criteria. As the adjacent motorized traffic volume and speed increases, or as the relative percent of heavy vehicles increases, bicyclists will try to move away from vehicles operating in the adjacent travel lane, positioning themselves closer to parked vehicles or the edge of roadway which can increase their crash risk.

Table 6-13. One-Way Standard Bike Lane Width Criteria

One-Way Standard Bike Lane Width Criteria		
Bike Lane Description	Desired Width (ft)	Constrained Width (ft)
Adjacent to curb[1] or edge of pavement	5 - 7	4
Between travel lanes or buffers	5 - 7	4
Adjacent to parking[2]	6 - 7	5
Intermediate or sidewalk level raised bike lane[1,2]	5.5 - 7.5	5
To allow side-by-side bicycling or passing	8 - 10	7

[1] The usable width of the bike lane which is measured from the outside lane stripe to either the gutter joint or 1' from the nominal face of a monolithic curb.

[2] Raised bike lanes adjacent to parking should have a minimum width of 7 feet

Signing and Marking

A solid white edge line (6-in wide) should be placed between the bike lane and travel lane. Lane lines between parked cars and the bike lane are optional.

Standard bike lane symbols and arrows, per the TMUTCD should be used to inform bicyclists and motorists of the restricted nature of the bike lane, and markings should be placed at periodic intervals to remind motorists of the presence of bicyclists.

Bike lane symbol markings should be placed no more than 50-ft downstream from an intersection. The first marking after an intersection or driveway should be placed outside of the wheel path of turning vehicles to reduce wear.

- In urban areas, it may be appropriate to space the symbols closer than 250-ft where motorist conflicts may be higher such as approaches to areas with significant parking turnover, intersections, driveways, or turn lanes.
- In suburban and rural areas, with long distances between intersections and little roadside activity, bike lane symbols may be spaced 1,000-ft apart or more.

Bicycle Design Speed

Not applicable as the roadway will be designed to accommodate motorist target design speeds which will exceed bicyclists.

Cross Slope and Grade - not applicable

Other Considerations

The effective width of a bike lane should not include rumble strips or standard drainage inlets.

Drainage grates located in the bike lane should be designed to prevent bicycle tires from catching in the grate pattern.

6.4.4.6 Raised Bike Lanes

A conventional bicycle lane can be raised above the street grade to create more separation from vehicles when a separated bike lane (see Section 6.4.4.3) with horizontal separation is not feasible or desired. In general, a separated bike lane is preferable to a raised bike lane to prevent motor vehicle encroachment and to reduce the potential for a bicyclist to crash while transitioning from the raised bike lane to the roadway at intersections. While a mountable curb between the raised bike lane and travel lane can reduce crash risk for bicyclists, it will not discourage motorists from encroaching into the raised bike lane. Raised bike lanes should not be installed adjacent to on-street parking due to the greater risk of dooring.

Raised bike lanes can be raised slightly, between street level and sidewalk height (intermediate level), or at can be located at sidewalk height (sidewalk level). Specific information on the curb types adjacent to raised bike lanes are discussed in Section 6.4.3.6.

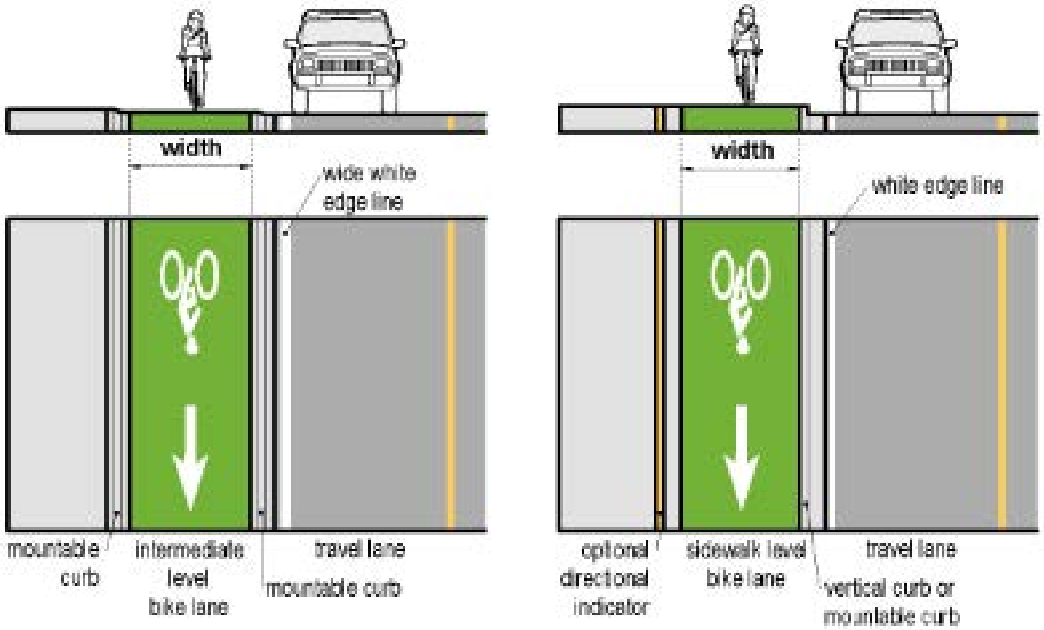


Figure 6-25. Raised Bike Lane Considerations

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.4.6.1 Basic Design Guidelines

Application

A raised bike lane can be paired in a corridor with either a separated bike lane or a conventional bike lane in scenarios where cross sections vary and/or additional conspicuity is desired for the bikeway users. These are most commonly used on roadways which do not have sufficient width to provide separated bike lanes with desirable street buffers where the alternative would be to provide a standard bicycle lane. Raised bike lanes require bicyclists to merge into the travel lane or a standard bike lane at intersections or to transition to a sidewalk (see Section 6.4.5.2),

Width

The width of raised bike lanes should accommodate the anticipated bicyclist demand and reduce the likelihood that a bicyclist will have to transition to an adjacent travel lane or sidewalk to pass other bicyclists or to avoided hazards such as debris, surface defects, or objects in the bike lane. The width should also consider the elevation of the bike lane.

Table 6-13 provides minimum and constrained widths for both raised bike lane scenarios.

Signing and Marking

A white edge line should be located at the edge of the travel lane to clarify the edge with the bike lane. A wide white edge line may be used, but it is recommended at locations with an intermediate height bike lane to provide additional emphasis of the mountable curb or the lower height curb (see Figure 6-25).

To further differentiate the raised bike lane from an adjacent travel lane or sidewalk and increase awareness of the elevation change the raised bike lane may:

- Be built with contrasting paving materials;
- Include a normal width white edge line adjacent to the sidewalk curb; and/or
- Include BIKE LANE (R3-17) regulatory signs at locations with a supplemental RAISED plaque in black letters.

Bicycle Design Speed

For raised bike lanes, a target design speed of 15 mph is generally appropriate as these bikeways are typically narrower than separated bike lanes and bicyclists do not have the ability to enter the roadway. Higher design speeds may introduce safety challenges where bicyclists interact with motorists and pedestrians. Lower bicyclist and motorist operating speeds at conflict points allow bicyclists and motorists more time to perceive potential conflicts. This is particularly important at intersections with raised bike lanes where bicyclists must merge into the roadway or enter a shared pedestrian space.

Cross Slope and Grade

A minimum cross slope of 1 percent typically provides adequate conveyance of drainage

Other Considerations

To prevent motor vehicle encroachment into the bike lane, a *sidewalk-level* raised bike lane built with a curb between the bike lane and travel lane is preferred. Where sidewalks are adjacent to the raised bike lane, a detectable edge should be provided to reduce the likelihood people with vision disabilities will enter the bike lane. This may lead to the construction of an intermediate-level bike lane. Section 6.4.4.3 provides additional guidance for sidewalk buffer design.

At locations where an *intermediate-level* raised bike lane is less than 7-ft in width, the bike lane should have a continuous mountable curb on both sides, between the bike lane and travel lane and the bike lane and the sidewalk, allowing bicyclists to traverse the curb if necessary. While the provision of a curb along the travel lane is more likely to discourage motorists from entering the raised bike lane, it may also decrease the comfort and safety of bicyclist if the bike lane is not sufficiently wide or if it is necessary for the bicyclist to exit the bike lane.

The bike lane elevation may vary within a single corridor via bicycle transition ramps which raise or lower the bike lane as needed at pedestrian crossings, transit stops, driveways, and intersections. Additional details on the design of intersections is discussed in Section 6.4.5 later in this guide. Frequent elevation changes along a corridor should be avoided because they reduce the comfort of the bicycling environment and can create maintenance challenges.

Raised bike lanes will require special considerations for maintenance activities as it may be difficult to maintain a debris free surface with standard street maintenance practices. For example, street sweepers cleaning an adjacent travel lane may push additional debris onto an intermediate level bike lane. Additionally, drainage will be based on the curb type used and may require inlets instead of drainage grates.

Raised bike lanes on roadways with frequent driveways should be considered carefully. The driveway design may require the bikeway to be lowered, resulting in frequent elevation changes which may be uncomfortable.

6.4.4.7 Bike Accessible Shoulders

6.4.4.7.1 Description

Figure 6-26. Example Bike Accessible Shoulder Schematic

Bike accessible shoulders are one-way facilities on a roadway that carry bicycle traffic in the same direction as adjacent motor vehicle traffic. A bike accessible shoulder is one that is at least as wide or wider than a bike lane to accommodate bicyclists and paved to provide a smooth, solid surface across its width. While the bike accessible shoulder distinguishes predictable areas for bicyclist and automobile movement, bicyclists may leave the shoulder to pass other cyclists or avoid debris and other traffic conflicts.

6.4.4.7.2 Basic Design Guidelines

Application

See Figure 6-6 in Section 6.4.2.4 for guidance on recommended shoulder width.

In rural environments, see Figure 6-6 in Section 6.4.2.4 for guidance on desirable shoulders widths.

Width

For any given roadway, the determination of the appropriate paved shoulder width should be based on the roadway's context and traffic conditions following the guidance in Chapter 3 of the RDM for shoulder requirements on new location and reconstruction projects. However, it should be noted that the shoulders in the RDM may consist of graded (soft) and paved (hard) surfaces. To ensure accommodation of bicyclists, it is desirable for paved shoulders to conform to the widths in Figure 6-6.

Some shoulders should be up to 10-ft wide adjacent to higher speed roadways as indicated in Figure 6-6 to allow bicyclists to operate with more separation to the high-speed traffic. Roadways indicated in TxDOT's Bicycle Tourism Trails Study[V] should be designed with a 10-ft shoulder (8-ft minimum) shoulder, a shared use path, or another locally preferred facility type should be available. The minimum widths for a bike accessible shoulder are defined below.

- A minimum width of 4' is allowable in low speed (45 mph or less) conditions.
- A minimum width of 5' is allowable for high-speed conditions.
- A minimum width of 5' is required for shoulders adjacent to bridge railings, MBGF, and other vertical elements.

The width of the bike accessible shoulder does not include rumble strips. See below for more information on rumble strip considerations.

Signing and Marking

A solid white edge line (6-in wide) should be placed between the shoulder and travel lane.

Bike accessible shoulders typically do not have bike lane markings, but they may include signage indicating the presence of bicyclists.

A bike accessible shoulder may transition to a bike lane with signage and markings at intersections to communicate the exclusive use of the space by bicyclists.

Bicycle Design Speed – not applicable

Not applicable as the roadway will be designed to accommodate motorist target design speeds which will exceed bicyclists.

Cross Slope and Grade

Cross slopes of 1 percent are more comfortable for people with disabilities and people bicycling with more than two wheels (e.g., cargo bike, adult tricycles, or trailers); however, cross slope may match existing roadway conditions without a maximum where necessary.

Other Considerations

Rumble strips are used to warn the driver that they are leaving the travel way and therefore may have a beneficial effect on the safety of bicycles using the shoulder. If rumble strips are to be used, or are anticipated in the future, allowances should be made in the shoulder to provide an adequate width for bike accommodations beyond the rumble strip. Profile pavement markings serve a similar function as milled rumble strips and can be considered as an option to avoid a reduction in the width of the accessible shoulder. Where bicycle traffic is expected, rumble strips should be designed as follows to minimize crash risk for bicyclists (see Figure 6-27). See latest RS Standards for additional guidance on rumble strips.

Periodic gaps (Figure 6-28) should be provided to allow bicyclists to safely enter or exit a shoulder as needed (e.g., to avoid debris, pass other bicyclists or disabled vehicles, make left turns, etc.) without having to ride over the rumble strips.

- Where bicyclists are operating at 20 mph or less, a minimum 15-ft gap every 40 to 60-ft allows half a second for a bicyclist to cross the rumble strip.
- Where bicyclists are operating over 20 mph, the gap should be increased to 20-ft or more or the rumble strips should be located on the right side of the shoulder to allow bicyclists to avoid them if they encounter a need to enter the travel lane (e.g. a downhill location).

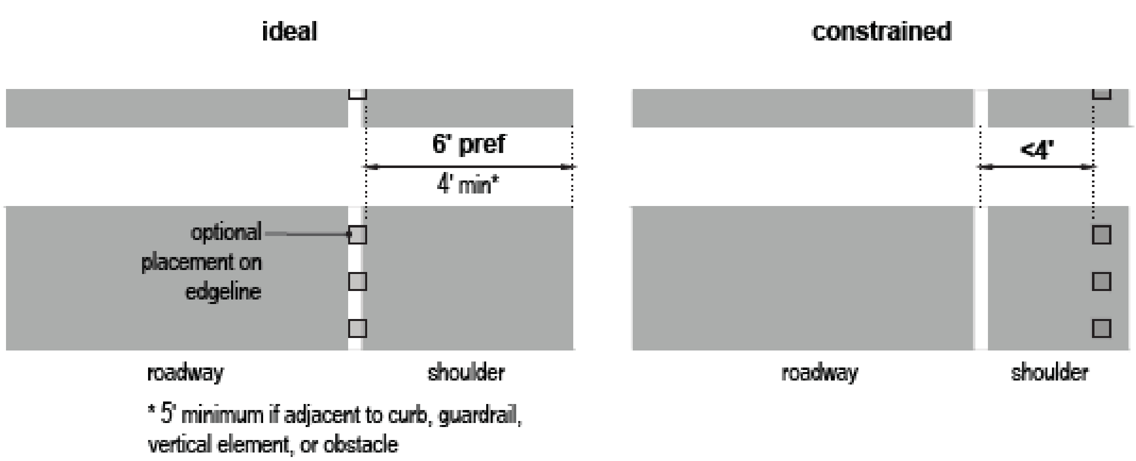


Figure 6-27. Rumble Strip Placement in a Shoulder

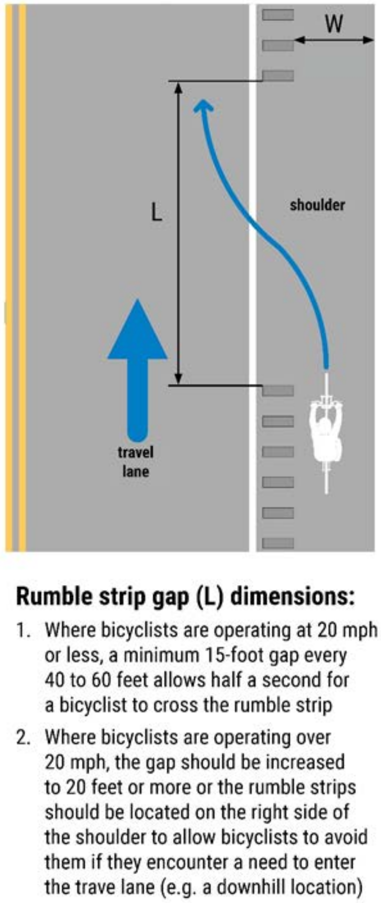


Figure 6-28. Rumble Strip Design and Gap Placement

6.4.4.8 Shared Lanes (wide outside lane)

6.4.4.8.1 Description

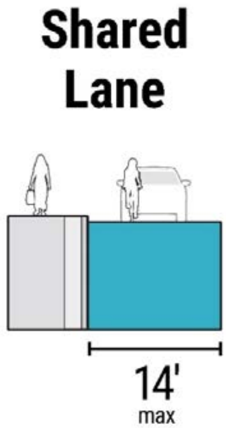


Figure 6-29. Example Shared Lane Schematic

Shared lanes (wide outside lane) are lanes that allow compatibility of operation for both motorized vehicles and bicycles. Since bicycles may be operated on all roadways except where prohibited by statute or regulations, shared lanes without markings already exist in many different urban, urban core, suburban and rural town settings.

Note that although marked shared lanes are allowed in the TMUTCD for certain conditions, TxDOT as a general policy does not recommend marked shared lanes for TxDOT roadways due to the higher speed nature of TxDOT roadways as compared to local jurisdictions.

6.4.4.8.2 Basic Design Guidelines

Application

Shared wide outside lanes in urbanized applications should only be used in locations with low volumes (3,000 ADT or lower) and low speeds (35 mph or less).

Width

The usable width for a wide outside lane should be 14-ft maximum and 13-ft minimum. The usable width is measured from the lane stripe to either the gutter joint or 1 -ft from the nominal face of a monolithic curb.

If the usable width is greater than 14-ft, a bike lane should be provided instead. Use of minimum travel lane widths may be necessary to incorporate the bike lane.

Signing and Marking

Typical supplemental signage may include: BICYCLES MAY USE FULL LANE (R4-11). See TMUTCD for signing applications.

Custom signage may include language instructing vehicles to change lanes to pass or use a 3-ft passing distance.

Bicycle Design Speed

Not applicable as the roadway will be designed to accommodate motorist target design speeds which will exceed bicyclists.

Cross Slope and Grade - not applicable

6.4.4.9 Rural Bikeway Types

Figure 6-30. Example Photo of Rural Shared Use Path

6.4.4.9.1 Shared Use Path

A Shared Use Path adjacent to roadway (sidepath) with separation from the roadway is an option on rural facilities. An additional option is a Shared Use Path on an independent alignment. While it is recognized that these types of facilities are not usually feasible on most rural projects, consideration should be given to using them on the Texas Bicycle Tourism Trails Example Network and rural roadways with ADT over 6000. If they are used, see Section 6.4.4.2 and the AASHTO Bike Guide for further design guidance.

Figure 6-31. Example Rural Bike Accessible Shoulder Schematic

6.4.4.9.2 Bike Accessible Shoulders

Bike accessible shoulders in rural areas function the same as bike accessible shoulders in urban areas with the exception that the roadway will generally not have curb at the edge. If they are used, see Section 6.4.4.7, and the AASHTO *Bike Guide* for further design guidance.

6.4.4.9.3 Shared Lanes (wide outside lanes)

Shared lanes (wide outside lane) are lanes that allow compatibility of operation for both motorized vehicles and bicycles. Since bicycles may be operated on all roadways except where prohibited by statute or regulations, shared lanes without markings already exist in many rural settings.

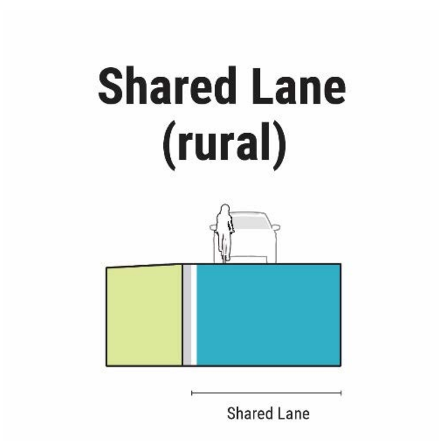


Figure 6-32. Example Rural Shared Lane Schematic

Note that although marked shared lanes are allowed in the TMUTCD for certain conditions, TxDOT as a general policy does not recommend marked shared lanes for TxDOT roadways due to the higher speed nature of TxDOT roadways as compared to local jurisdictions. Also, shared lane markings alone do not provide any additional safety for bicyclists and do not substantiate a dedicated bicycle facility. In a rural applications, shared wide outside lanes should only be used in locations with low volumes (1,000 ADT or lower) and speeds of 45 mph or less.

6.4.5 Intersections and Crossings

The design of bikeways at intersections will come in many configurations as they are natural locations to transition between bikeway types and to respond to conditions where bikeways are discontinued. Each intersection is unique and requires engineering judgment to determine an appropriate design to maximize safety. The following guidance provides examples for how to transition bikeways at intersections for common conditions.

Due to the mixed nature of traffic at intersections (pedestrians, bicyclists, and motorists), the designer should keep in mind the speed of each travel mode and its resulting effect on design values when considering design treatments. The fastest vehicle should be considered for approach speeds (typically the bicyclist and motor vehicle) because these modes require the greatest stopping distance. By contrast, for departures from a stopped condition, the characteristics of slower users (typically pedestrians and bicyclists) should be considered due to their greater exposure to cross traffic.

Intersection crossings occur within the functional area of an intersection of two or more roadways (see Figure 6-33). Intersection crossings for bicyclists are typically parallel to at least one roadway and have unique operational challenges. Geometric design guidance for intersections should also be applied to driveway crossings and alley crossings to promote safety and legibility for bicyclists and pedestrians. Mid-block crossings are those located outside of the functional area of any adjacent roadway intersection. Grade-separated crossings pass over or under a roadway and eliminate conflicts between bicyclists and motorists.

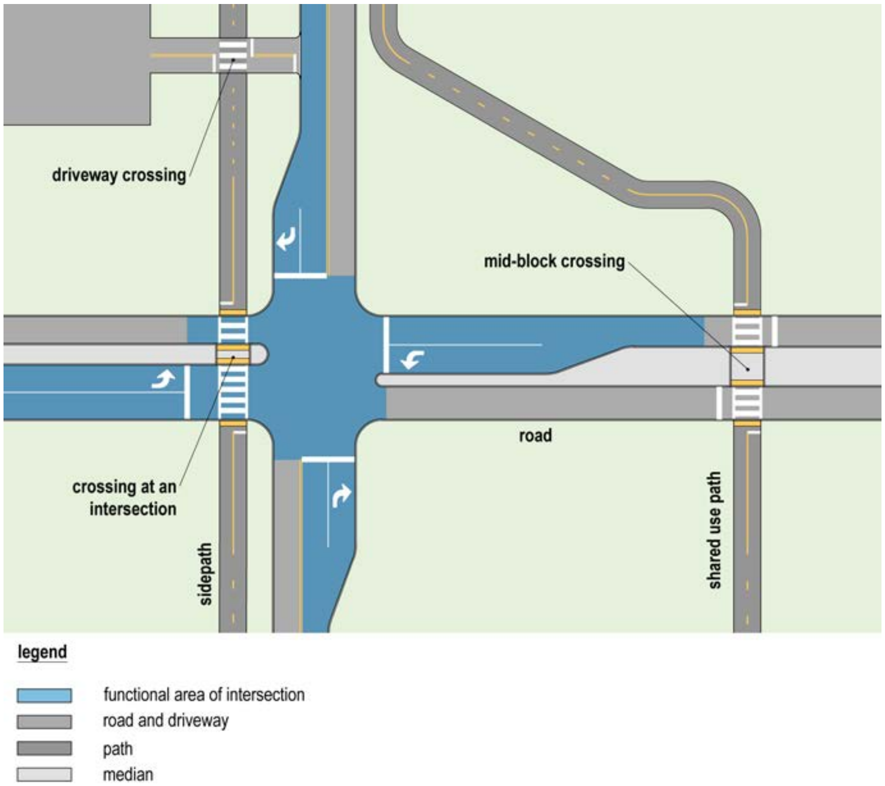


Figure 6-33. Crossing Locations Relative to Intersections Functional Area

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.1 Principles of Intersection Design

6.4.5.1.1 Minimize exposure to conflicts

Intersections with bikeways should be designed to minimize bicyclist exposure to motorized traffic and minimize bicyclist conflicts with both motorists and pedestrians. Exposure to conflicts can be eliminated with:

- geometric design treatments;
- restricting turn movements;
- providing traffic signal phasing that manages conflicts; or
- providing grade separation where appropriate.

Where elimination of conflicts is not possible or practical, intersection designs should limit the amount of time and space where bicyclists are exposed to moving or crossing traffic in locations where:

- Bicyclists cross multiple vehicular travel lanes,
- Bicyclists operate between moving vehicular travel lanes,

- Bicyclists wait in areas exposed to moving motor vehicle traffic (e.g., waiting to turn left in a shared lane),
- Motorists merge with, or turn across the path of, bicyclists, and
- Bicyclists cross pedestrian travel paths or other bikeways.

While they do occasionally occur, crashes between bicyclists and pedestrians are comparatively rarer than those between bicyclists and motorists or between pedestrians and motorists. Designers can minimize crash risk between bicyclists and pedestrians by providing clear sight distance between pedestrians and approaching bicyclists at locations where bicyclists cross a pedestrian facility.

6.4.5.1.2 Reduce speeds at conflict points

If conflict points cannot be eliminated, intersection design should minimize the speed differential between users at the points where travel movements intersect by reducing motorist speeds at these locations. Intersections where bicyclists operate should be designed to ensure slow speed motorist turning movements (10 mph or less) and weaving movements (20 mph or less) across the path of bicyclists.

6.4.5.1.3 Communicate right of way priority

Intersection design should provide bicyclists, pedestrians, and motorists with cues that both clearly establish which user(s) have the right of way and consistently communicate expected stopping or yielding behavior.

Traffic control devices should communicate the priority right of way through the provision of:

- Marked crosswalks at shared use path crossings;
- Marked bicycle crossings of roadways and driveways;
- Providing audible and vibrotactile devices for people with disabilities at crossings of bikeways where appropriate;
- Marked pedestrian crossings of bikeways;
- Regulatory or warning signs for crossing, merging, or turning traffic where appropriate; and
- Signalization where appropriate.

6.4.5.1.4 Provide adequate sight distance

It is necessary to provide adequate sight distances and visibility between bicyclists, motorists, and pedestrians as they approach intersections. Adequate sight distance is needed to perceive and react to avoid potential conflicts. See Section 6.4.3.4 for more details.

6.4.5.1.5 Provide clear transition between bikeway types

Intersections are likely to be locations where bicyclists transition between different types of bikeways. These transitions should be intuitive to all users of the intersection. It is also important to provide clear and direct paths for pedestrians across bikeways. See Section 6.4.5.2 for more details.

6.4.5.1.6 Accommodate people with disabilities

Intersections should be designed in accordance with accessibility guidelines. Designs should ensure that people with limited or no vision are given sufficient cues to prevent them from unintentionally moving into the street or a bike-only bikeway. See Chapter 7 Section 3.2.2 and other general information in the pedestrian guidance.

6.4.5.2 Intersection Approach Treatments

Managing potential conflicts with merging or crossing motor vehicle traffic is a fundamental challenge for bicyclists operating within bikeways at intersections. As a bikeway approaches an intersection, designers should aim to provide a continuous and direct route through the intersection, driveway, or alley that is legible to all users of the roadway. Designers should also aim to minimize or eliminate conflict areas and to maintain bikeway continuity up to and through an intersection, as intersections can be the most stressful locations for bicyclists to navigate. Constrained environments may occur at intersections where additional motorist capacity is needed to accommodate turn lanes. Designers have several options when balancing bikeway designs with the need to accommodate turning traffic. Given the operational reality that bicyclists are operating on the right-side of the roadway as a slower, vulnerable roadway user, a key design influence is the interaction of bicyclists with right turning traffic and the geometric design of the intersection where this interaction occurs. The design of the intersection approach is influenced significantly by the bikeway design on the approach and departure of the intersection. The following guidance covers the most common transition scenarios.

For all scenarios described in this section, a high-comfort optional bicycle ramp to a sidewalk or protected intersection (see Section 6.4.5.3) should be considered where the bikeway selection guidance in Section 6.4.2.4 indicates a need for additional separation and the existing condition is a shared lane, shoulder or bicycle lane.

6.4.5.2.1 Shoulder or Bicycle Lane Terminated to a Shared Through Lane

Figure 6-34 shows an intersection where a shoulder or bicycle lane is terminated because the roadway narrows, or the lane becomes an additional through travel lane. The bicycle lane or shoulder should be dotted between 50 and 200-ft prior to the point where it ends to clarify this is a location where bicyclist must either merge into the traffic lane or where a bicycle ramp is provided, have the option to exit to a sidewalk, separated bike lane, or sidepath. A BICYCLE warning sign (W11-1) or BICYCLES MERGE sign may be considered at the start of the dotted lane line followed by a

BICYCLES ON ROADWAY or BIKES MAY USE FULL LANE sign. At locations with bicycle lanes, a BICYCLE LANE ENDS sign may be posted in lieu of a BICYCLES MERGING sign.

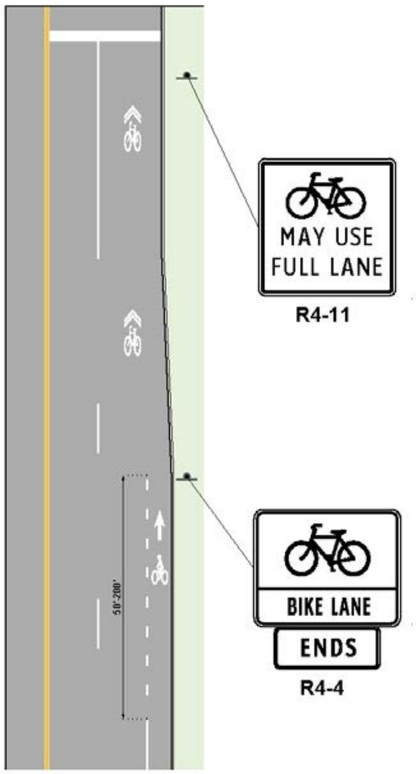


Figure 6-34. Example Shoulder or Bike Lane Terminating to a Shared Through Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.2.2 Shoulder or Bicycle Lane Terminated to a Shared Right Turn Lane

Where there is insufficient space for a bike lane and a right turn only lane, designers must assess the risk to bicyclists when considering which of the following is a preferred option based on operational conditions at the location:

- bike lane transitions to a shared right-turn lane (Figure 6-35)
- bike lane transitions to a shared through lane (Figure 6-36)
- bike lane transitions to a protected intersection (Figure 6-41) or sidewalk via a bike ramp

Shared lane markings may be located within the left side of the turn lane (Figure 6-35) if the lane is a minimum 14-feet, otherwise the shared lane markings should be located within the center of the right turn lane or located within the adjacent through travel lane (Figure 6-36) following TMUTCD guidance. Designers should consider the volumes for each movement, the queuing length, time spent queuing, and operational speeds when determining if bicyclists should be placed in the through or right-turn lane.

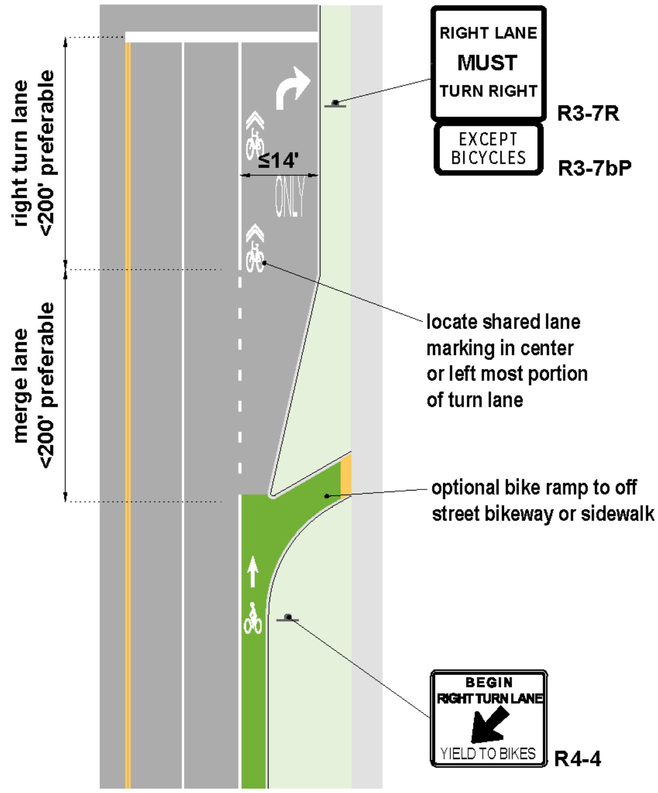


Figure 6-35. Example Bike Lane Transitions to a Shared Lane Markings in Right Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

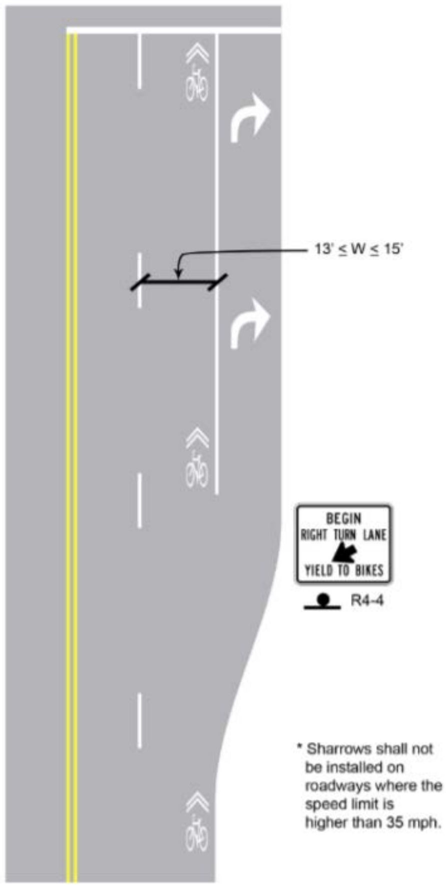


Figure 6-36. Example Bike Lane Transitions to a Shared Lane Markings in Through Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.2.3 Shoulders and Bicycle Lanes Continue with Addition of Right Turn Lanes

Figure 6-37 shows an intersection where a through travel lane becomes a right turn only lane. In this scenario, the bike lane should shift to the left of the turn lane. The bike lane should remain along the curb until it is within 400-ft of the intersection. The bike lane drops at this point and is re-introduced on the left side of the right turn lane within 200-ft of the intersection.

In this scenario, the bike lane should not be striped diagonally across the travel lane, as this inappropriately suggests to bicyclists that they do not need to yield to motorists when moving laterally. Where operating speeds are less than 35 mph, shared lane markings may be used to delineate the likely path of travel of bicyclists transitioning across the shared lane between the bike lanes. A BICYCLE warning sign (W11-1) or BICYCLES MERGE sign should be placed where the curb side bike lane ends.

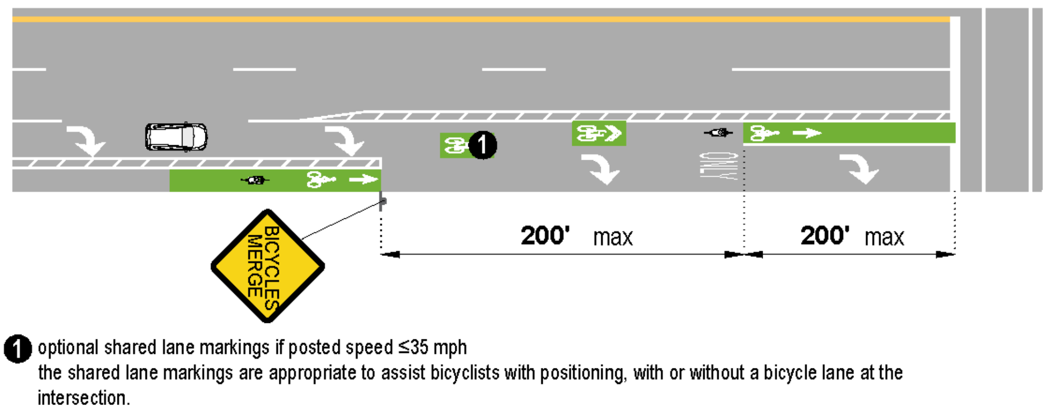


Figure 6-37. Bike Lane Transitions to Through Bike Lane with Right Turn Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

Figure 6-38 shows an intersection that transitions a bike lane to a separated bike lane on the right side of right-turning vehicles. This may be desirable at locations where there are more than 150 right turning vehicles during the peak hour or an existing crash history. Signal phase separation of bicyclists and motorists should be considered with a bicycle signal. In this scenario, it is preferable the separated bike lane provide:

- A 6-ft bike lane width desirable exclusive of a gutter, 4-ft minimum; and
- A 6-ft buffer width desirable, 2-ft minimum.

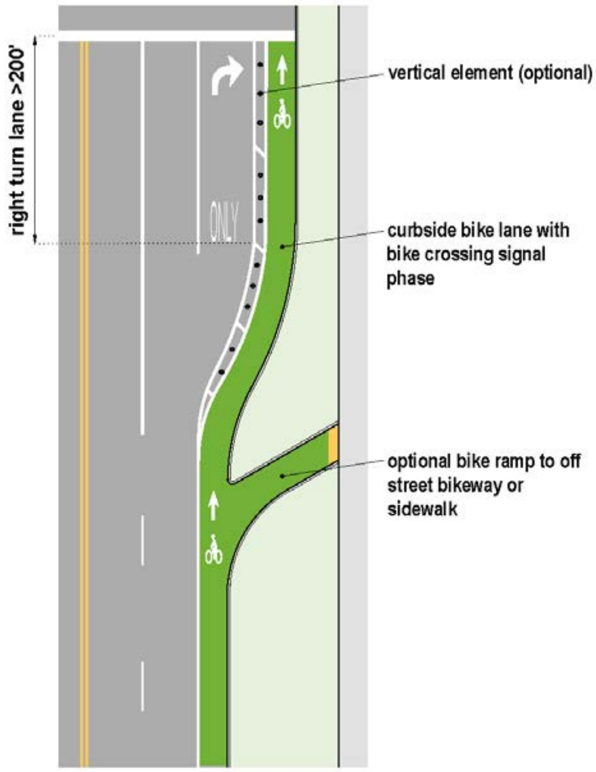


Figure 6-38. Example Bike Lane Approach to a Right Turn Only Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

Figure 6-39 shows a right turn lane which requires drivers to physically cross the bike lane within a constricted area formed by vertical elements, such as medians or flexible delineators to force motorists to enter the turn lane at a clearly defined location, thus providing a more predictable conflict point. The opening must be designed to accommodate the desired motorist operating speed. A target speed of 35mph or less is preferred for the use of this design.

In this scenario it is preferable the separated bike lane provide:

- a 6-ft bike lane width desirable, 4-ft minimum; and
- a 2-ft minimum buffer width is required; otherwise provide maximized width bike lane (Figure 6-39)

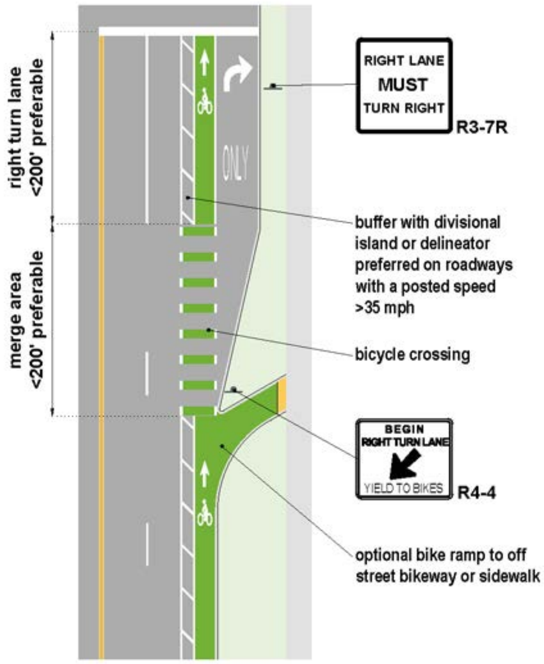


Figure 6-39. Bicycle Lane with Constricted Motorist Entry Point

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

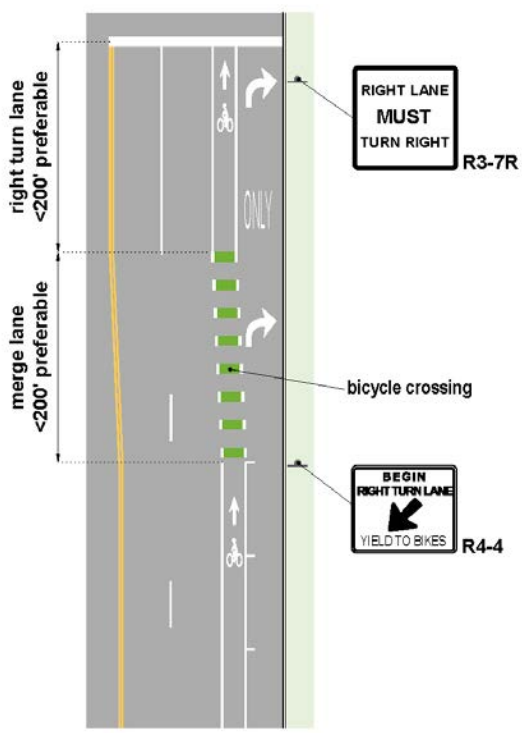


Figure 6-40. Standard Bicycle Lane with Right Turn Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.2.4 Shoulder Transition to Bicycle Lane

Designers can transition a shoulder to a bicycle lane prior to intersections where it is desired to emphasize bicyclists have the right-of-way at the intersection as through moving vehicles, and then transition back to a paved shoulder on the far side of the intersection (Figure 6-41). Transitioning a paved shoulder to a bicycle lane at intersections may be desirable at locations near high-speed exit and entrance ramps (greater than 35 mph) where shoulders are the predominant bicycle accommodation in the in the area.

Where shoulders are a part of a bike route, the striping should not taper towards the cross street at intersections, but it should transition to a dotted edge line where motorists are expected to use the shoulder to begin their turning movement (see Figure 6-41). The right edge of the shoulder for the turning roadway can be marked with a solid white line where delineating this edge is important for safety reasons.

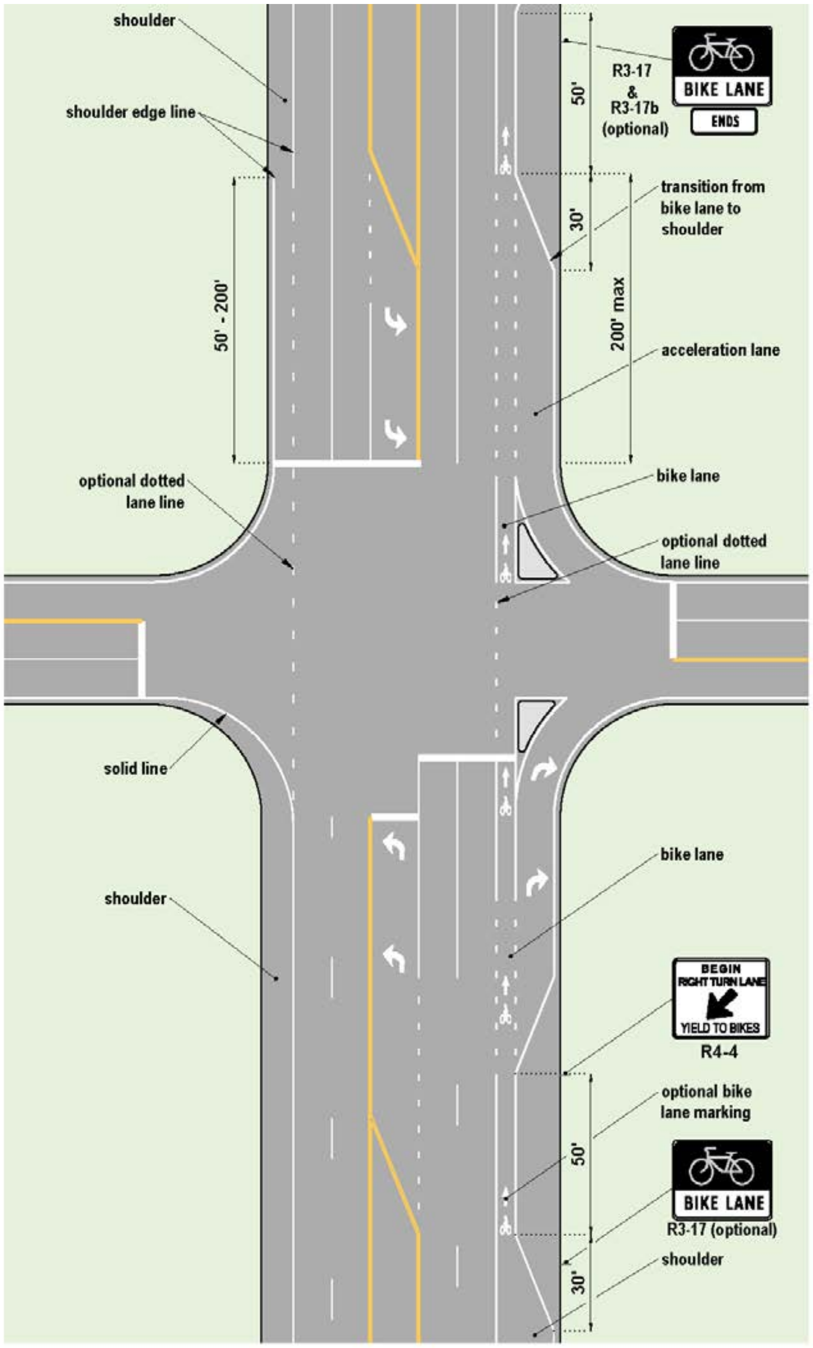


Figure 6-41. Example Shoulder Markings to Accommodate Bicycling

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.2.5 Shoulder Transition to Bicycle Lane at T-Intersections with Bypass Lanes

Bypass lanes at T-intersections of two-lane roadways can be designed to facilitate the passing of motorists stopped to make left turns onto intersecting roads without encroaching into a bicyclists operating space (Figure 6-42). Where this is done on a roadway with paved shoulders, at least 4-ft of shoulder pavement should be carried through the intersection along the outside of the bypass lane and designated as a bike lane. This is especially critical on roadways with higher volumes and operating speeds where bicyclists operating on the shoulder are likely to be in conflict with traffic using bypass lanes.

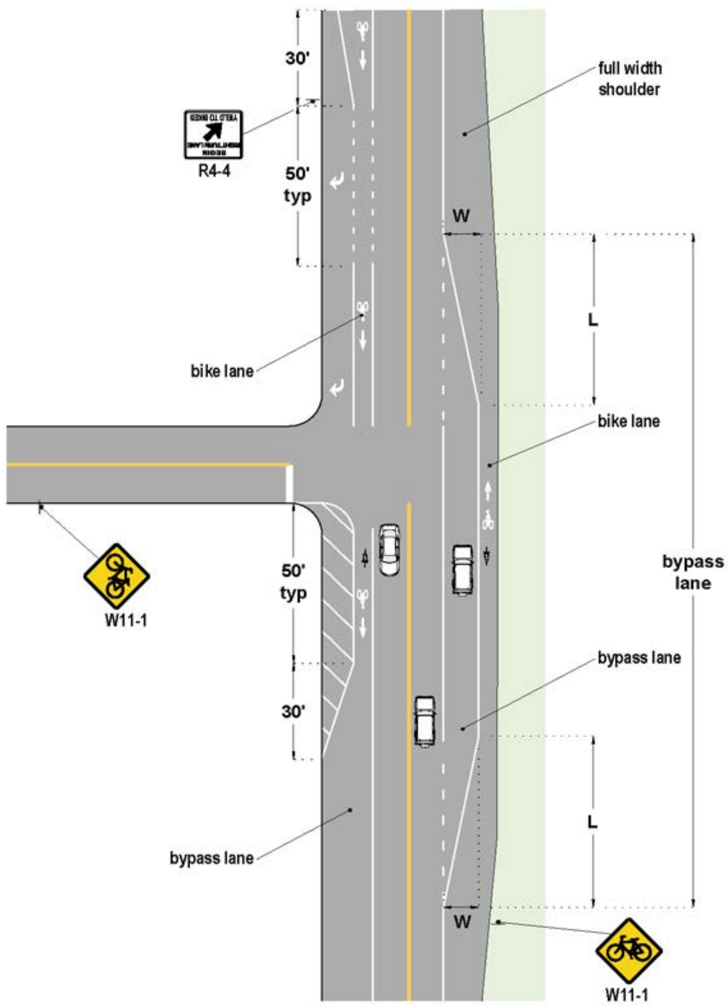


Figure 6-42. Motorist Bypass Lane with Bicycle Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.2.6 Protected Intersection or Continuation of Separated Bike Lane or Sidepath

Protected intersections result from the application of a separated bike lane or sidepath up to the intersection (Figure 6-43). They may also be developed by transitioning a shared lane, shoulder, or bicycle lane into a separated bike lane or sidepath in advance of an intersection. They may be used at signalized and unsignalized intersections and driveways.

The defining features of protected intersection are a recessed bicyclist crossing and intersection corner geometry designed to slow right turning motorist speeds to improve motorist stopping or yielding to through moving bicyclists and parallel crossing pedestrians. Designers should aim to create an offset between the adjacent vehicle lane and the bike crossing between 6-ft and 16.5-ft from the adjacent motor vehicle lane. This treatment has been shown to significantly reduce crashes at uncontrolled and permissive conflict locations.

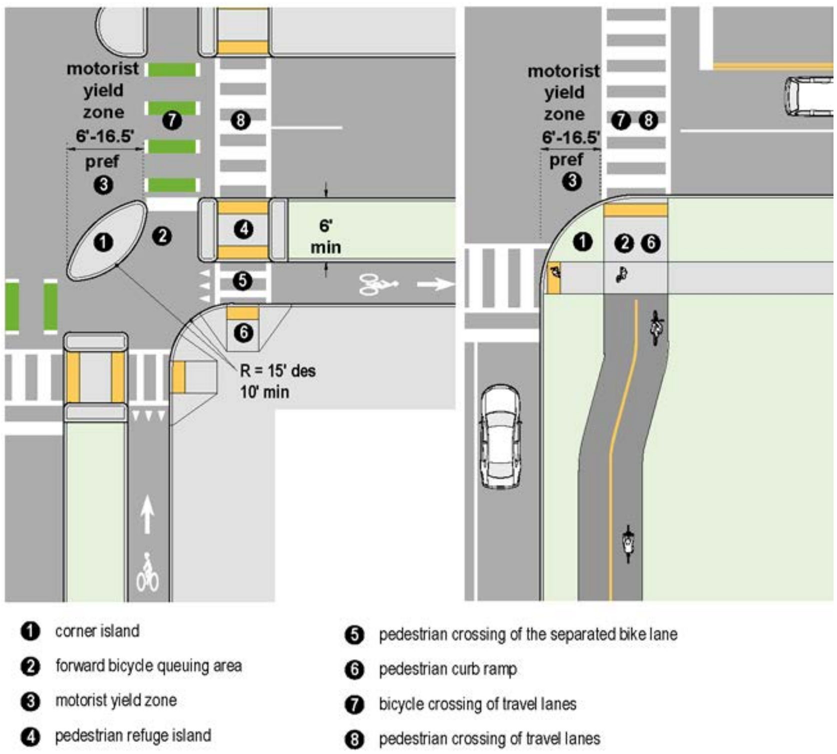


Figure 6-43. Protected Corner Treatment and Intersection Design Components

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

Corner Island

A key component of a protected intersection is the corner island, which provides the following benefits:

- Is used as a design control to set the horizontal offset between the adjacent motor vehicle lane and the bike crossing, to create a motorist stop or yield zone;
- Is used as a design control to slow turning vehicles to a desired turning speed in combination with truck aprons where necessary;
- Positions bicyclists waiting to cross in front of adjacent stopped motorists improving their visibility to motorists;

- Creates queuing space for bicyclists making a two-stage turn, outside of the path of through bicyclists; and
- Allows for a pedestrian refuge island, shortening their crossing and reducing exposure.

A corner island is a raised island between the bike lane and travel lane that defines the motorist's turning radius and slows their turning speed. They are typically constructed using concrete and curbing. They may be constructed with low-cost materials, such as paint and flexible delineator posts or engineered rubber curbs and/or rubber speed cushions. Figure 6-44 shows details for corner islands in a protected intersection.

When on-street parking is located along the corridor, parking restrictions at intersection approaches will provide space for a street buffer to install a corner island. When there is no parking along the corridor, the bike lane can be offset away from the travel lane to create space for the desired corner island and motorist stop or yield zone. The corner island geometry will also impact the size of the motorist stop or yield zone and can be designed to slow approaching bicyclists' speeds at the point of potential conflict by requiring some deflection of the bicyclists' travel path. That deflection should generally not exceed the width of the bicycle lane.

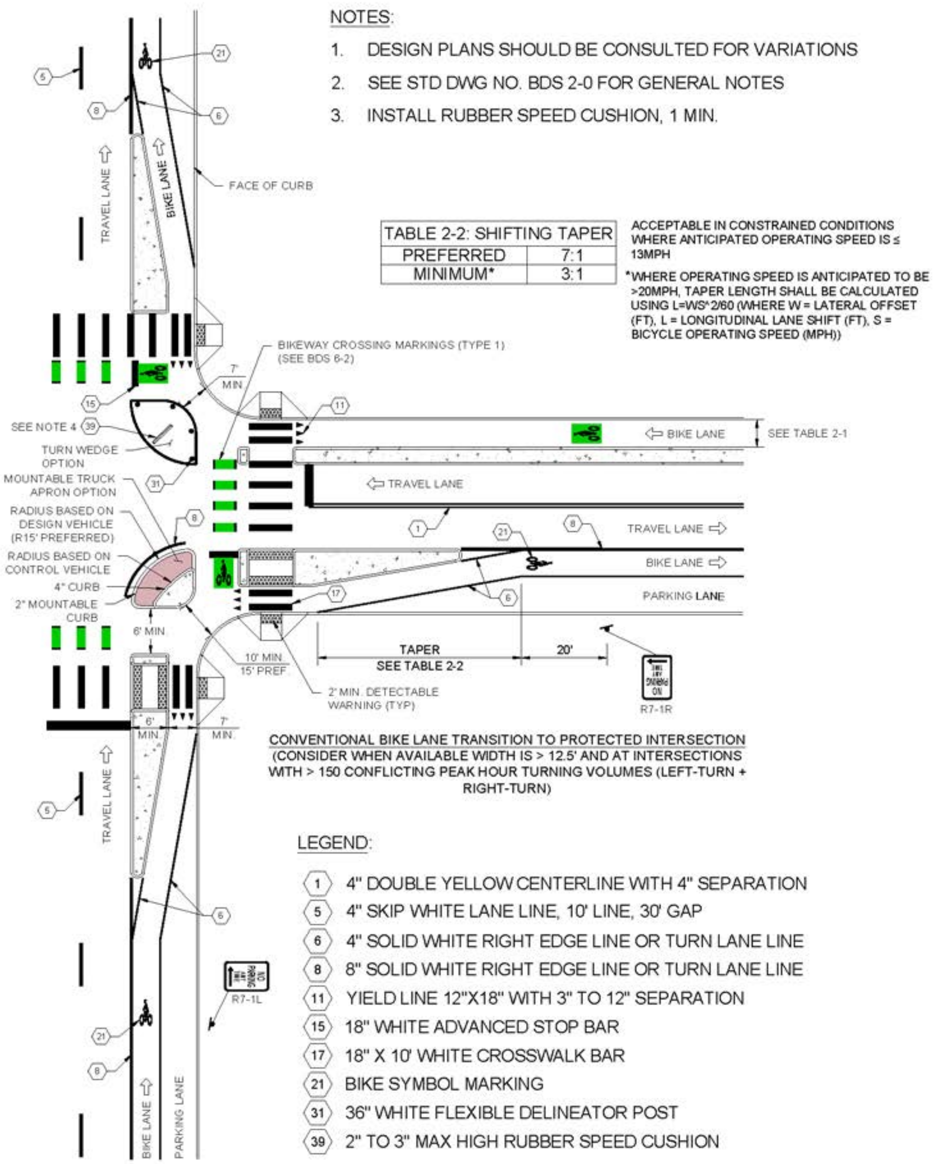


Figure 6-44. Protected Intersection Treatments with Truck Apron

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

Truck Aprons

A truck apron is a design feature of some corner islands and is designed to accommodate the turning needs of large vehicles while slowing the turning speeds of smaller vehicles. The truck apron portion of the corner island is designed to be mountable by larger vehicles (the design vehicle) to accommodate their larger effective turning radius. The outside edge of the truck apron is constructed with a mountable curb (i.e. the interface between the pavement and truck apron), and should be designed for a passenger (P) or delivery vehicle (SU-30), to turn outside the apron at a slower speed. Figure 6-45 below shows the larger vehicle's effective turning radius traversing the truck apron and the passenger vehicle's actual radius at the edge of the truck apron, noting that the bicycle stop bar is located behind the larger vehicle turning path.

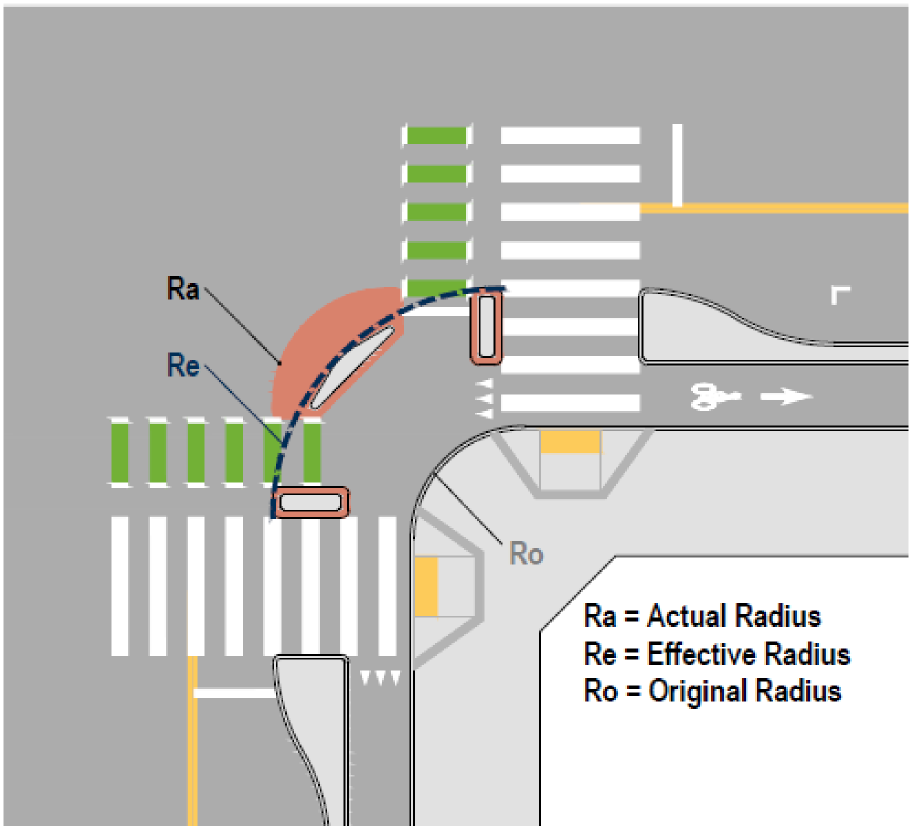


Figure 6-45. Actual vs Effective Radius

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

Pedestrian Considerations

When the street buffer is at least 6-ft in width, it may be used as a pedestrian refuge island, which shortens the crossing distance. In this case, pedestrians would cross the separated bike lane at an uncontrolled crossing, then cross the motor vehicle lanes as a separate crossing. When the crossing is located at a signalized intersection, the designer can consider reducing the signal timing for the pedestrian crossing to reflect this shorter crossing distance if the push buttons are located within the pedestrian refuge median. Stop bar markings (and associated R1-5b sign) and crosswalk markings should indicate the right of way between bicyclists and pedestrians at these locations.

When the street buffer is less than 6-ft in width and there is not room for a pedestrian refuge island, the crossing distance cannot be shortened, and the associated signal timing (if signalized) must be calculated for the entire street width.

Figure 6-46. Protected Corner Treatment Details: Truck Apron (left) and Turning Wedge (right)

Traffic Control Considerations

At all intersections, the designer should consider using a TURNING VEHICLES STOP TO PEDESTRIANS AND BICYCLES sign to communicate where turning motorists need to stop and yield to these street users. Use of a modified sign similar to the TURNING VEHICLES STOP FOR PEDESTRIANS (R10-15a) sign with a bicycle symbol may be considered.

In scenarios where a separated bike lane approaches an intersection that does not have adequate space for a protected intersection, the separated bike lane will have to transition to a bike lane or shoulder in advance of the intersection with a bicycle ramp.

6.4.5.2.7 Transitions from Protected Intersection to Other Bicycle Facility Types

When a protected intersection is provided for any bikeway type, the preference is for that protection to continue through the intersection and for the transition to a bikeway with less separation to occur on the far side of the intersection as shown below with the exception of sidepath transitions which require special considerations for pedestrians.

Farside Transition from Separated Bike Lane to Shared Lane

The transition into a shared lane should occur away from the crossed roadway to allow bicyclists time to stop and ascertain when it is safe to enter the roadway without conflict from approaching motorists who may be turning from the crossed roadway or approaching from the main roadway (Figure 6-47). This location should ensure a clear sight line between all approaching motorists and the bicyclist. It may be desirable to add a BICYCLES ON ROADWAY or BIKES MAY USE FULL LANE sign at the location where the bicyclist enter the roadway. A yield sign may be considered at the point where the bicyclists enters the roadway.

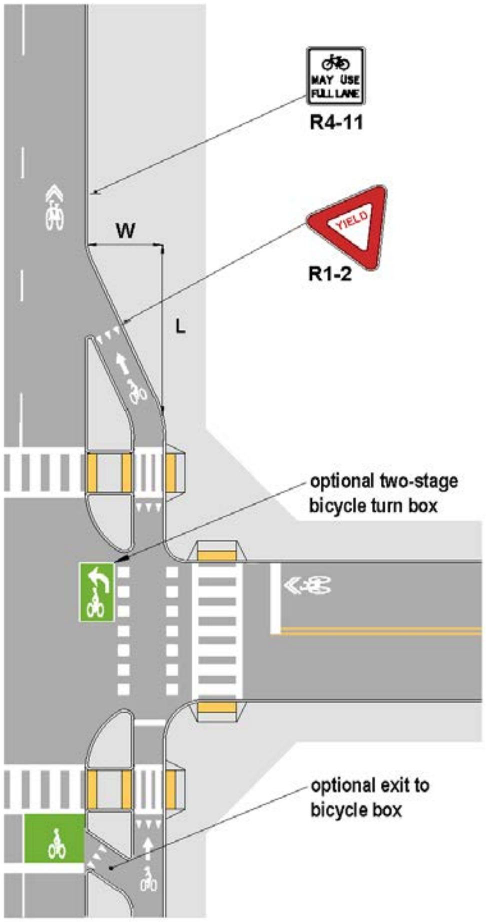


Figure 6-47. Transition to Shared Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

Farside Transition from Separated Bike Lane to Bicycle Lane or Shoulder

The transition into a bicycle lane on the far side of the crossing may occur at any point after the bicyclist crosses the pedestrian crossing (Figure 6-48). This will minimize the pedestrian crossing of the roadway.

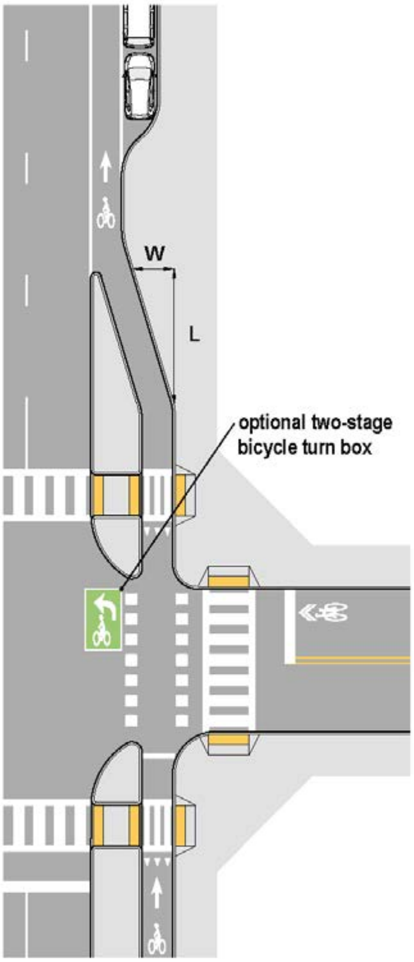


Figure 6-48. Transition to Bike Lane

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

Transitions from Two-Way Separated Bike Lanes to One-Way Separated Bike Lanes

Transitions of two-way separated bikeways into bikeways with one-way operation require additional considerations. Bicyclists operating in the contraflow direction will be required to cross at least two directions of travel and failure to provide a clear transition to the desired location may result in wrong way bicycle riding as shown in Figure 6-49 and Figure 6-50. The crossing may warrant bicycle signals at signalized crossings (see Section 6.4.3.8).

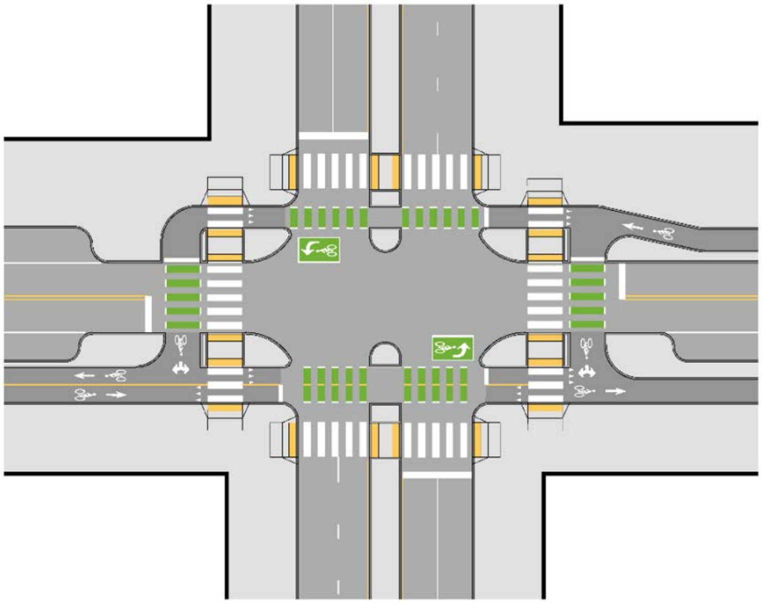


Figure 6-49. Transition from One-Way to Two-Way Separated Bike Lanes with Protected Intersection

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

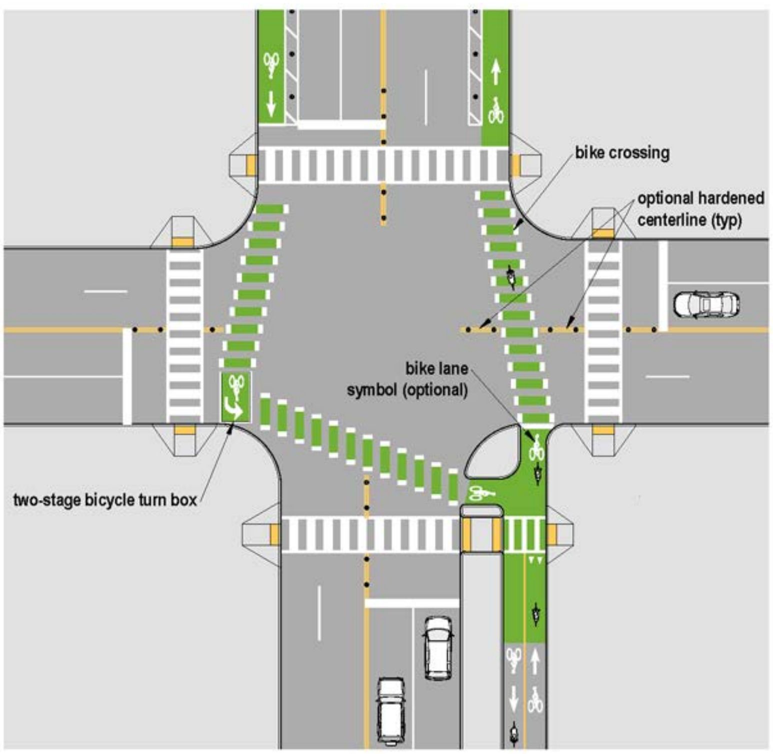


Figure 6-50. Transition from One-Way to Two-Way Separated Bike Lanes with Protected Corner and Two-Stage Turn Queue Box

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.2.8 Raised Bike Lane Considerations at Intersections

At locations where bike lanes are raised and located adjacent to a travel lane, the raised bike lane must transition to street level into a bike lane or shared lane, or it must bend away from the travel lane into a sidewalk, sidepath, or separated bike lane (see Figure 6-51). It is preferable to transition the raised bike lane to bend away from the travel lane to form a protected intersection which minimizes conflicts with turning motorists (Option 1).

Where protected intersections are not feasible, the raised bike lane should transition to street level. At intersections with low volumes of right turning traffic (less than 50 per hour), it is preferable for the raised bike lane to continue to the intersection and return to street level on a ramp within 10-ft to 30-ft of a pedestrian crosswalk (Option 2).

Where it is determined to transition the raised bike lane to a right turn lane, standard bike lane or shared lane, that transition should occur between 50-ft to 200-ft prior to the intersection (Option 3). At locations with higher volumes of right turning traffic, a bike ramp should be considered to allow bicyclists to transition to the adjacent sidewalk or sidepath.

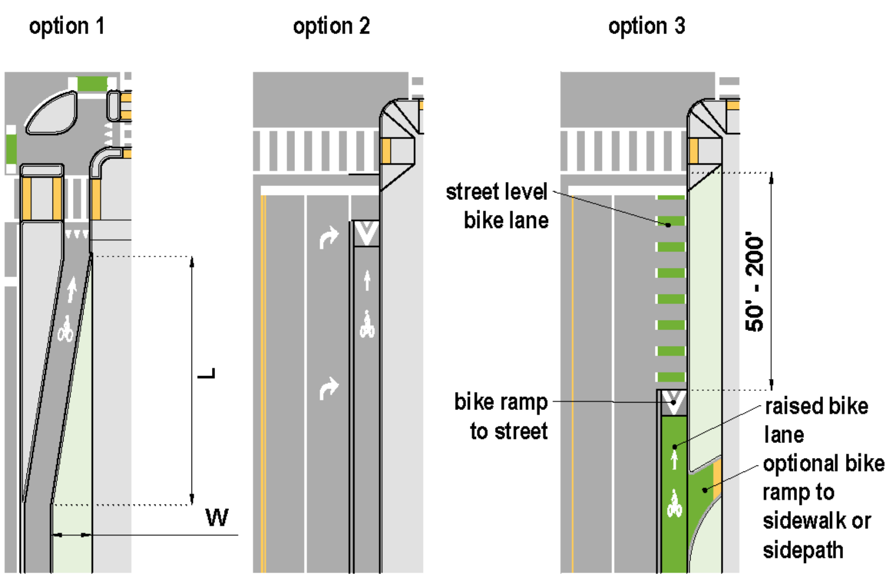


Figure 6-51. Raised Bike Lanes at Intersections

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.3 Bicycle Ramps to Transition Between Bicycle Facilities

Bicycle ramps can be used to transition bicyclists between on road bicycle facilities (shared lanes, bicycle lanes, and shoulders) to off road facilities (separated bike lanes, sidewalks, and sidepaths) as discussed in previous guidance. Bike ramps have commonly been used on the approaches to and departures from roundabouts.

It is preferable to desirable to design bike ramp transitions to occur with a gradual taper as depicted in Detail 1 of Figure 6-52 following the guidance of Section 6.4.3.5.

In constrained conditions, a more abrupt shift as depicted in Detail 2 of Figure 6-52 may be considered. These constrained bike ramps can present the following challenges:

- Narrow bike ramp widths can force bicyclist to encroach on adjacent motorist travel lanes, pedestrian zones, or on-coming bicycle traffic on two-way facilities in order to navigate the ramp; and
- If grade breaks at the top and bottom of the bike ramp are not perpendicular to the bicyclist path of travel, bicyclists with more than two wheels (i.e. adult tricycles, bikes with trailers, etc.) can experience instability or overturning.

In both situations, increasing the width of the bike ramp can help to address these issues.

Bike ramps are intended for the exclusive use of bicyclists and therefore need not comply with pedestrian accessibility guidelines, but grades similar to pedestrian curb ramps can help to address issues of comfort. Where the bike ramp connects directly to a sidewalk or shared use path, a detectable warning surface should be used at the top of the bike ramp.

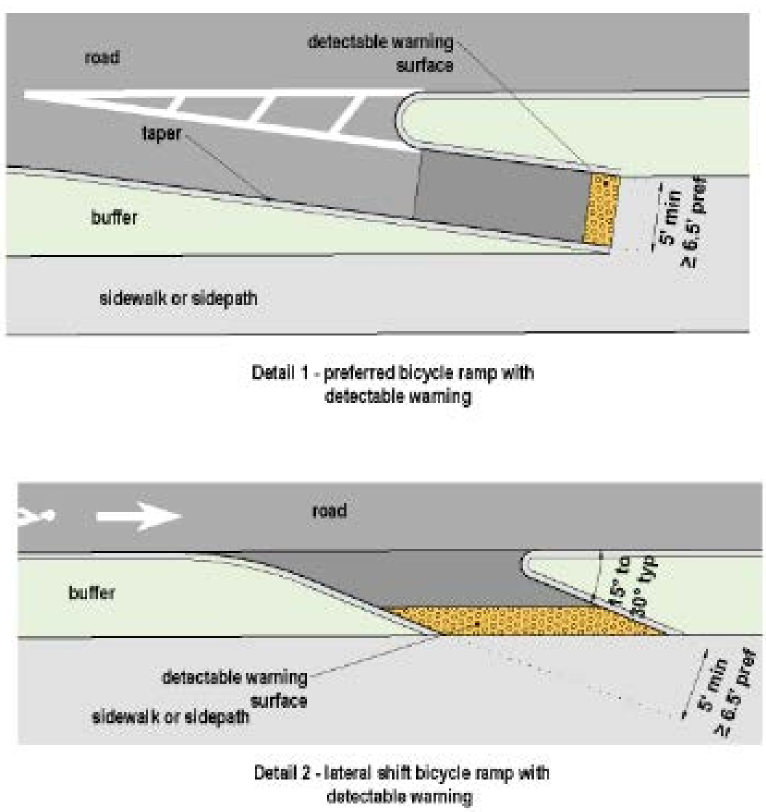


Figure 6-52. Bicycle Ramps

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.4 Driveways

6.4.5.4.1 Driveway Crossings of Shoulders and Bicycle Lanes

When on-street shoulders or bicycle lanes cross driveways, their pavement markings should extend through the conflict point as follows:

- Low Volume Driveways (<25 veh/day): Shoulder and bicycle lane markings should be maintained across driveway with solid white lines. Stop or yield control is optional for the motorists approaching the crossing from private property.
- Medium Volume Driveways (25-500 veh/day): Bike Crossing Markings should be used (see Section 6.4.3.8); Stop or yield control should be considered for the motorists approaching the crossing from private property.
- High Volume Driveways (>500 veh/day): Bike Crossing Markings should be used (see Section 6.4.3.8); Traffic control should be evaluated with an engineering study.

6.4.5.4.2 Driveway Crossings of Separated Bicycle Lanes and Sidepaths

For separated bicycle lanes at street level, vertical elements such as flex-posts should be used to control turning motorist speeds into and out of the driveway. They should be placed to define the driveway limits while accommodating the expected turning requirements of the design vehicle. At higher medium to high volume driveways, consideration can be given to developing a raised crossing. See Chapter 7 Section 3.6.3.4 and Figure 6-53 below for details on raised crossings.

For separated bike lanes and sidepaths at sidewalk level, that level should be maintained through the driveway crossing to maintain the raised crossing.

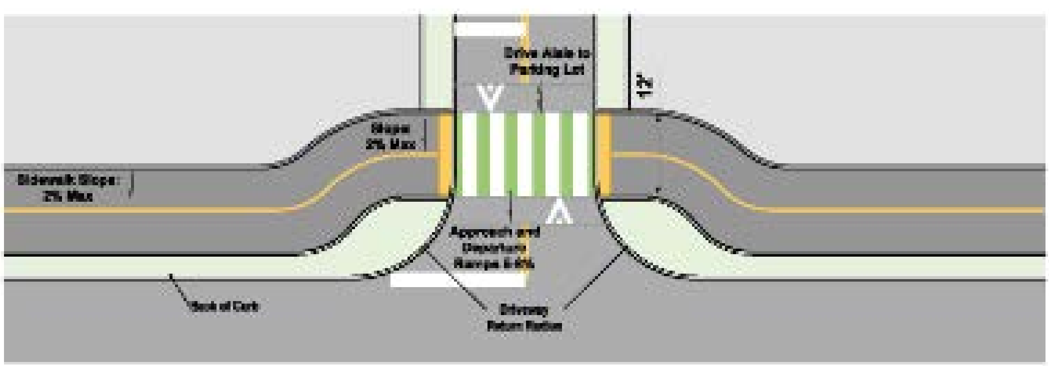


Figure 6-53. Raised Driveway Crossing

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

For all scenarios, adequate sight distances should be provided to ensure motorists exiting the driveway can see oncoming bicyclists, pedestrians, and motorists before leaving the driveway, and that motorists entering the driveway can see bicyclists and pedestrians approaching the driveway entrance and stop or yield appropriately. On-street parking, especially if it is serving as the vertical element for a separated bike lane, may need to be restricted on either side of the driveway to achieve desired sight lines.

Designers have two options for evaluating the sight distance. They must consider if motorists will take a two-stage approach and be permitted to block a separated bike lane or sidepath to view approaching motor vehicle traffic, or if there is a need for drivers to exit the driveway in a single movement. The single-stage scenario may be appropriate in locations where the motorist would otherwise block the bike facility for extended periods of time or where bicycle volumes or motorist volumes are anticipated to be high.

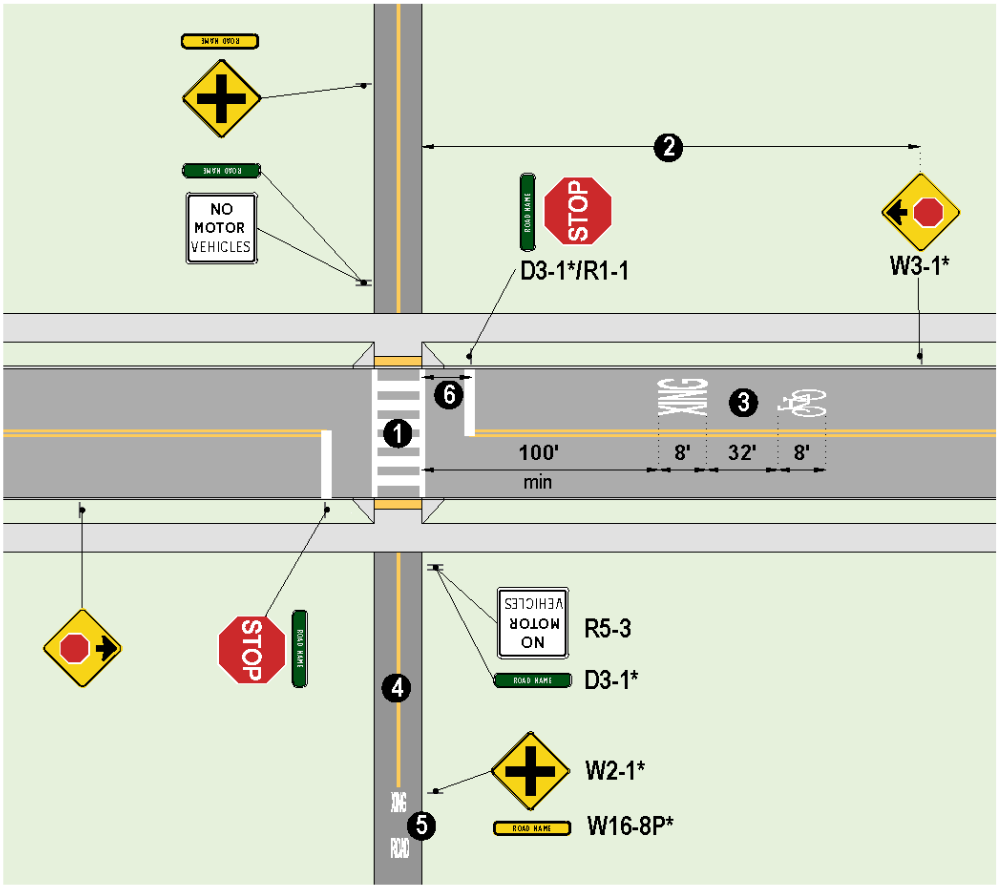
As discussed in Section 6.4.3.4, Chapter 2 Section 4, Intersection Sight Distance establishes recommended sight triangles that will generally result in sufficient sight distance for bicyclists.

6.4.5.5 Mid-Block Shared Use Path Crossings

It is preferable for mid-block crossings to intersect the roadway at a 90 degree angle to minimize the crossing distance. Mid-block crossings may be controlled (signalized) or uncontrolled (signing and markings only). Designers should consider traffic volumes and speeds of motorists, sight distances, the bicyclist target design user profile, and the walking speed of pedestrians when designing mid-bock crossings. This is particularly important for uncontrolled crossings because the crossing users are reliant on motorist stopping and yielding or finding gaps in traffic. The FHWA Guide to Improve Uncontrolled Crossings (STEPS) provides recommendations for appropriate treatments to consider for uncontrolled crossings, which can be used for shared use path crossings. Additionally, TMUTCD signal warrants may be used (with bicyclists counted as pedestrians) to identify if a full signal is appropriate. The signal warrant should consider the anticipated volumes of pathway users during the peak riding period (which may be on weekends).

Figure 6-54 through Figure 6-56 illustrate various examples of mid-block crossing control treatments. These figures show typical pavement marking and sign crossing treatments. The diagrams are illustrative and are not intended to show all signs and markings that may be necessary or advisable, or all types of design treatments that are possible at these locations. Each Figure assumes the appropriate minimum sight distances are provided for the roadway and the shared use path. See the FHWA STEPS guide for recommended treatments and countermeasures to address motorist stopping and yielding.

See the AASHTO *Bike Guide* for additional guidance for shared use path crossings.



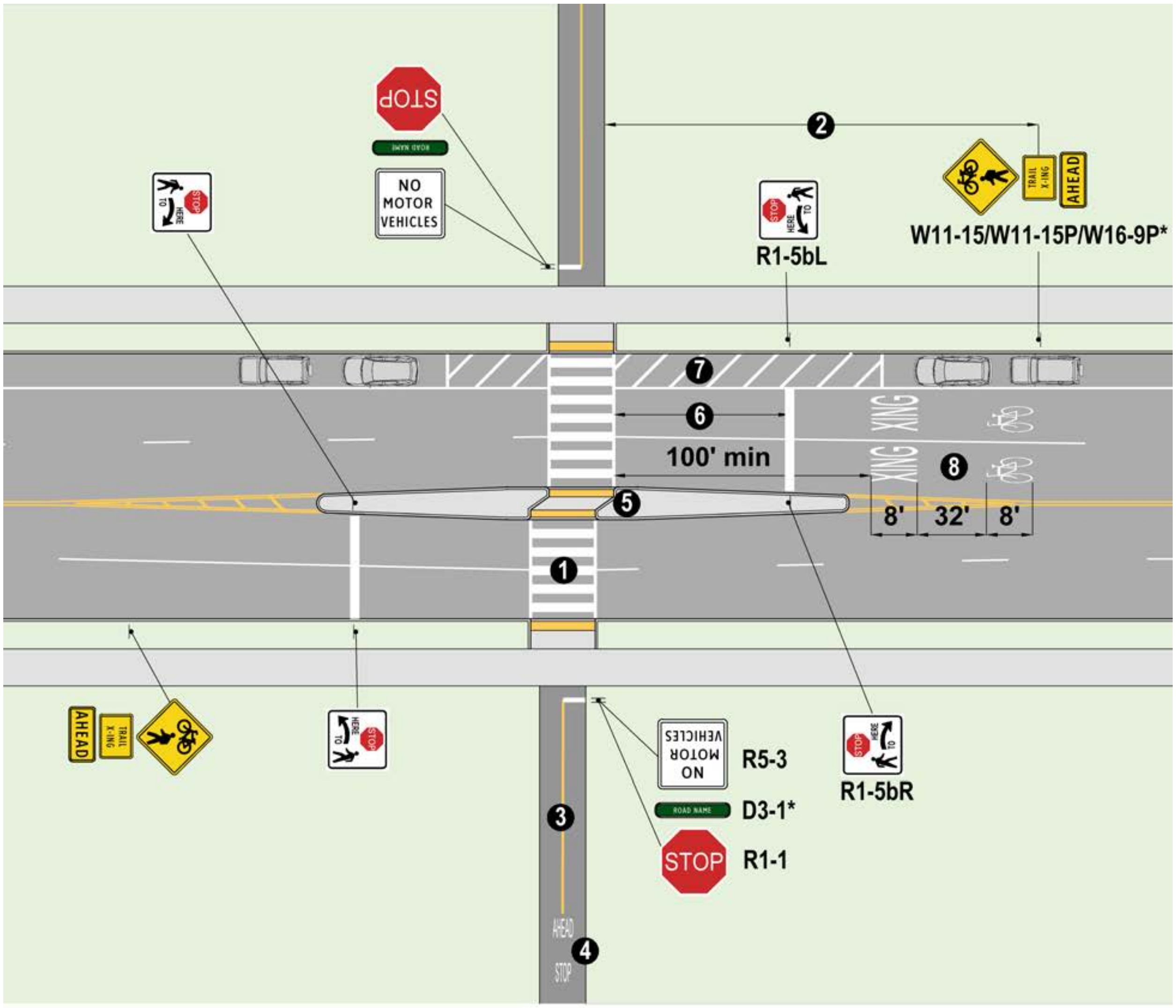
Note:

* **Signs are Optional but Recommended**

1. crosswalk markings legally establish midblock pedestrian crossing
2. length varies: see Texas MUTCD table 2C-4
3. optional roadway markings
4. shared-use path centerline as needed
5. optional pathway markings and signage
6. sign placement 4'-10' from crossing

Figure 6-54. Road Stops

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)



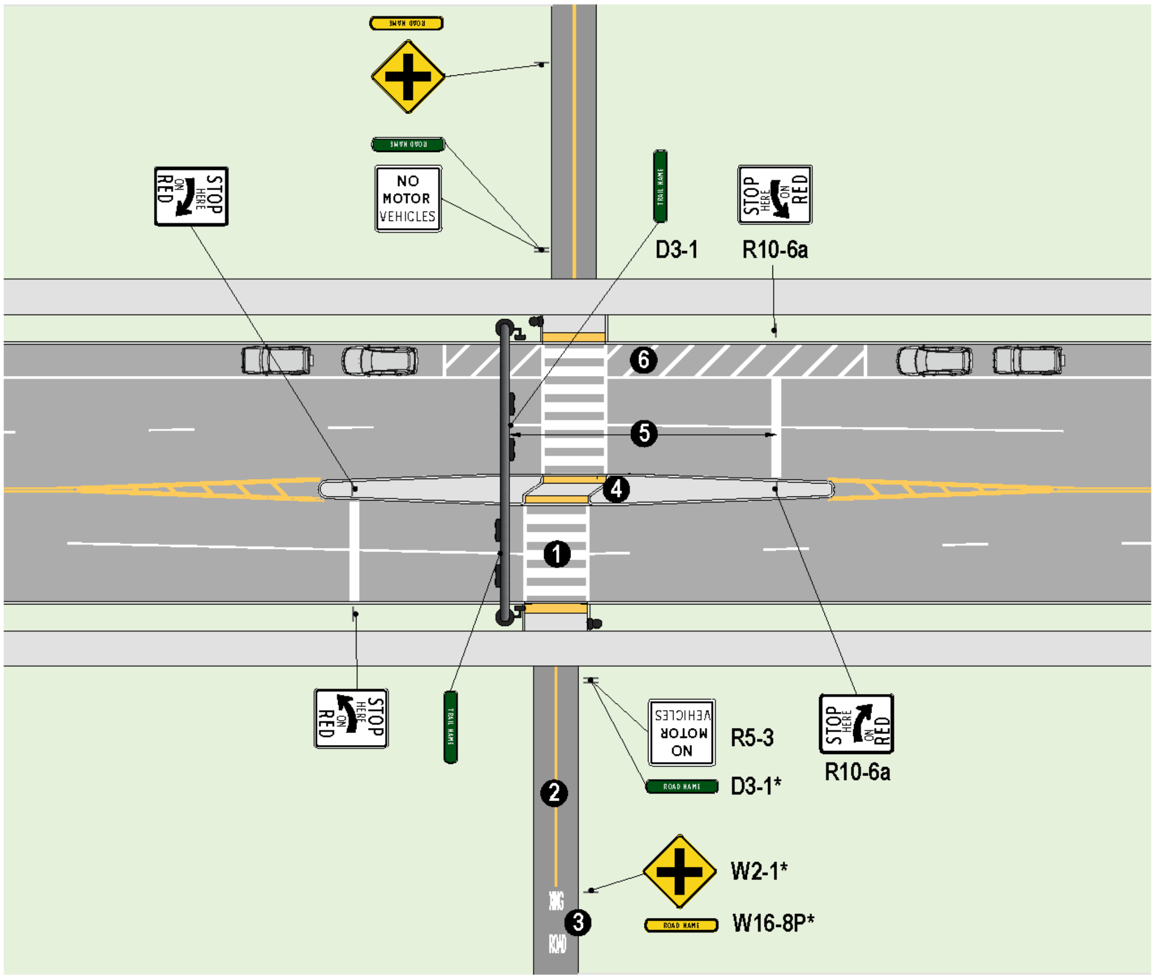
Note:

* Signs are Optional but Recommended

1. crosswalk markings legally establish midblock pedestrian crossing
2. length varies: see mutcd table 2C-4
3. shared-use path centerline as needed
4. optional pathway markings and signage
5. refuge median
6. stop bar placement 20'-50'
7. parking restricted
8. optional roadway markings

Figure 6-55. Multilane Road Uncontrolled with Advance Stop Bar Line

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)



Note:
* Signs are Optional but Recommended

1. crosswalk markings legally establish midblock pedestrian crossing
2. shared-use path centerline as needed
3. optional pathway markings and signage
4. refuge median
5. stop bar placement 40' min from traffic signal
6. parking restricted

Figure 6-56. Multilane Road with Signals

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.5.6 Roundabouts

Appendix E Section 2 provides guidance on roundabouts, which provide non-signalized traffic control at intersections with a circulating lane (or lanes) around a central island traveling counterclockwise.

While the Highly Confident bicyclist may be comfortable traversing a roundabout in a shared lane environment, many bicyclists will not feel comfortable navigating roundabouts with vehicular traffic, especially multilane roundabouts. Per the TMUTCD, standard bike lanes are not permitted to be located within the circulatory roadway of a roundabout. For comfort and safety reasons, roundabouts may be designed to facilitate bicycle travel outside of the circular roadway on a separated bike lane or shared use path.

This transition from on-road to separated bikeway should be located a minimum of 100-ft from the edge of the roundabout circulatory roadway (see Figure 6-57 and Figure 6-58). If on-street bike lanes are present, they should be terminated in advance of the roundabout at the transition to the separated bikeway. As shown on Figure 6-57, a bicycle ramp should be provided to transition between an on-street bikeway and a raised bikeway, such as a shared use path. Designers should provide an appropriate taper of the bike lane to narrow the entry width for the roundabout to only the motorist travel lanes. The bike lane line should be dotted for 50 to 200-ft in advance of the taper to provide guidance to bicyclists who wish to travel the roundabout in the shared lane.

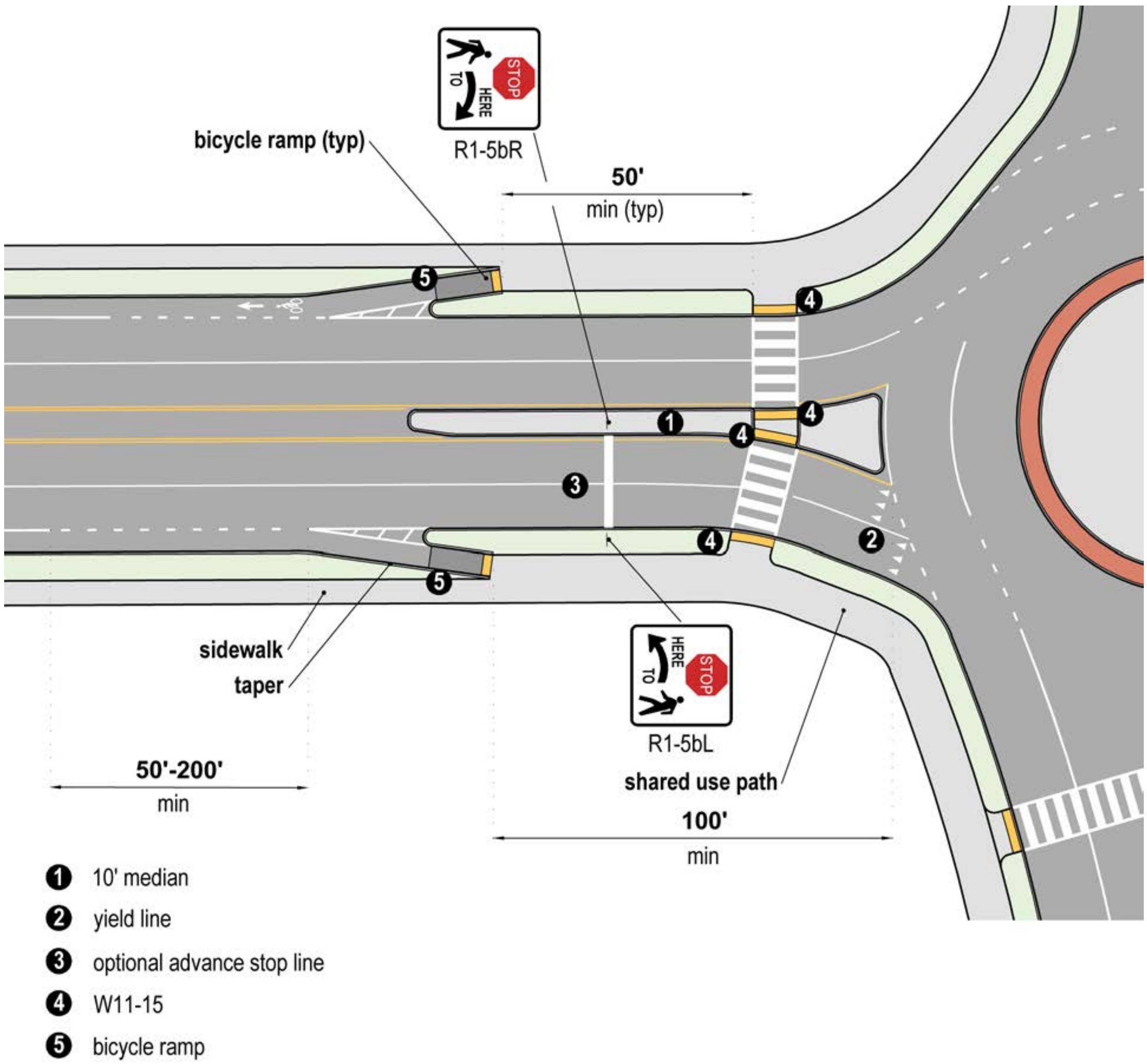


Figure 6-57. Typical Layout of Bike Lane Transitions to Shared Use Path at Multilane Roundabout with Bike Ramps

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

When separated bike lanes are provided on approaches to roundabouts, they may be continued around the intersection by maintaining their separation from the roadway to maintain the continuity of the bikeway When bike lanes are provided on approaches to roundabouts, and if it is desirable to maintain separation between bicyclists and pedestrians, the bike lanes may transition to separated bike lanes around the roundabout. Standard bicycle lanes are not allowed within roundabouts. Separated bike lanes at roundabout crossings should provide the following features:

- Stop and yield control for motorists at the bicycle and/or pedestrian crossing;

- Channelizing islands or detectable surfaces to maintain separation between bicyclists and pedestrians throughout the crossings;
- BICYCLE/PEDESTRIAN WARNING (W11-15) signs at the bicycle and pedestrian crossings; and
- STOP HERE FOR PEDESTRIANS AND BICYCLES (R1-5b modified) sign, supplemented with stop bars may be considered for bike crossings at roundabout exits to reinforce motorist stopping and yielding.

Additional signs or pavement markings may be appropriate to reinforce the bicyclist's and motorist's responsibility to stop and yield.

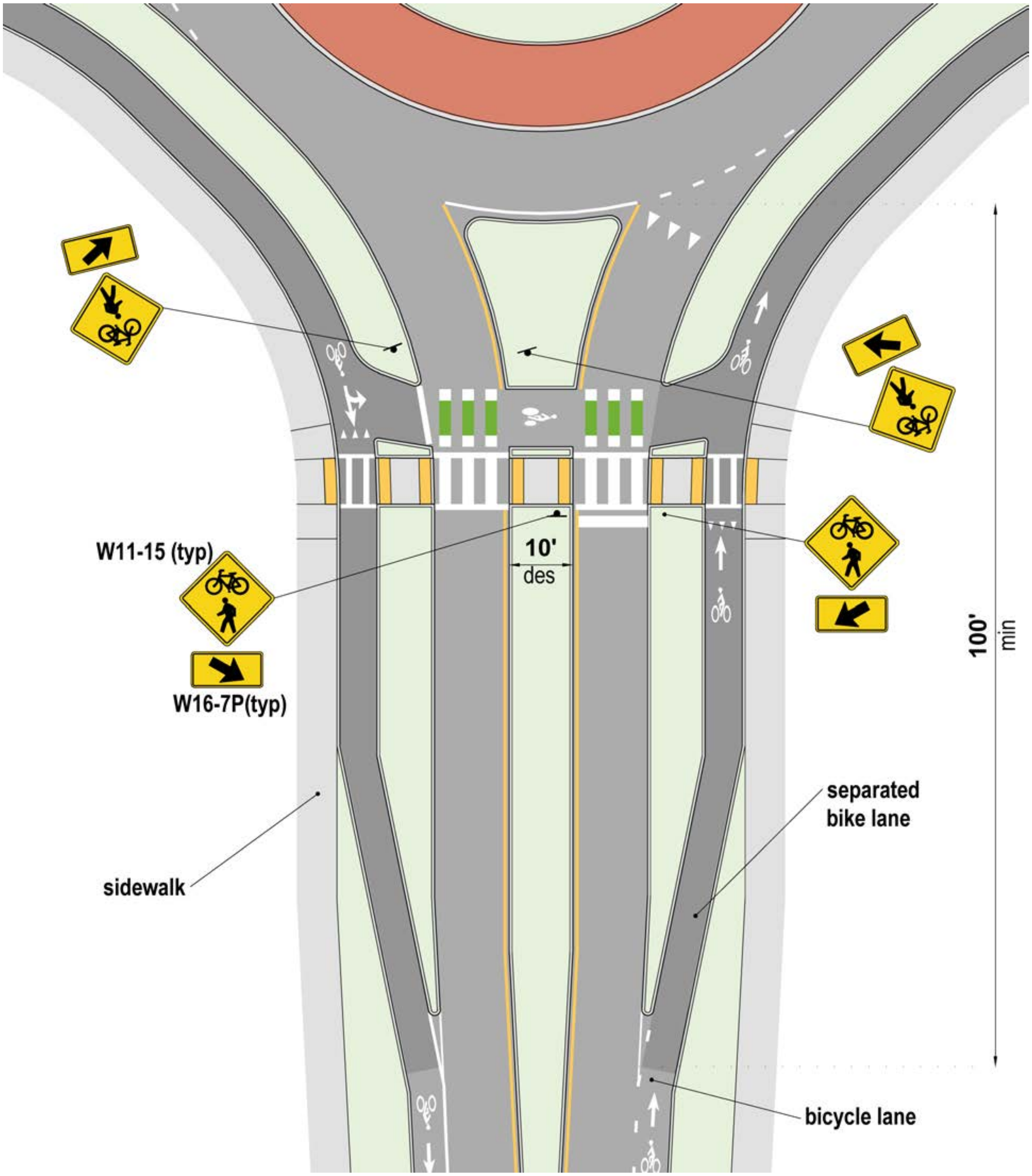


Figure 6-58. Typical Layout of Separated Bike Lanes at Roundabout

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

When shared use paths are provided approaching a roundabout, they should be continuous around the circulating roadway. Shared use path design at roundabouts is similar to separated bike lane design, and should include the following features:

- Minimum shared use path width of 10-ft;
- Widened curb ramps that match the shared use path width at crosswalks to facilitate pedestrians and bicyclists at the crossings;
- Supplemental stop bars for crossings at roundabout exits to reinforce motorist stopping and yielding;
- BICYCLE/PEDESTRIAN WARNING (W11-15) signs at the shared use path crossings; and
- Bicycle crossing/conflict markings to delineate the bicyclist path of travel may be provided.

6.4.5.7 Railroad Crossings

Railroad tracks that interface with bikeways can be hazardous to bicyclists. The following design considerations are important for bikeways located near tracks:

- Design the bicycle crossing of the tracks to be between 60 degrees and 90 degrees;
- Provide, as practical, the best track surface treatment for bicyclists;
- Provide firm, stable, and slip-resistant pavement;
- Reduce the flangeway width;
- Provide clear delineation with pavement markings that indicates to bicyclists where they should travel to cross railroad tracks at an optimal location;
- Provide warning signs to alert bicyclists of the crossing; and
- Provide adequate sight distances for approaching bicyclists to see the crossing ahead.

Generally, there are two conditions for bikeways and railroad interfaces: tracks located parallel to the bikeway, and tracks located crossing the bikeway.

6.4.5.7.1 Parallel Tracks

Tracks that are located parallel to a bikeway can be located in the street (e.g., streetcar) or adjacent to the roadway. These tracks are potentially hazardous to bicyclists, both when travelling parallel to the track and when crossing over the track.

While traveling parallel to the track, a bicyclist's wheel may fall into the flangeway if they travel too close to the track. This may be particularly noteworthy where bikeways are located in the street, in close proximity to rail tracks. In this case, the bikeway should be clearly delineated, and the distance between the bikeway and the track should be maximized.

Bicyclists may need to cross parallel tracks when turning onto or off of the corridor. When turning right, bicyclists should be encouraged (through pavement markings) to cross the track at an angle between 60-degrees and 90-degrees. This reduces the chance of a bicycle wheel falling into the flangeway and the chance of a bicyclist slipping on the rail. If tracks are located to the left of the bikeway, bicyclists should be encouraged to use a two-stage turn box to turn left so that they cross the tracks at a 90-degree angle. Figure 6-59 shows the design detail of a bikeway crossing parallel tracks.

6.4.5.7.2 Crossing Tracks

Railroad tracks that cross a street typically cross at an angle near 90-degrees. If the crossing angle is skewed, Figure 6-59 shows an example of a bikeway crossing railroad tracks.

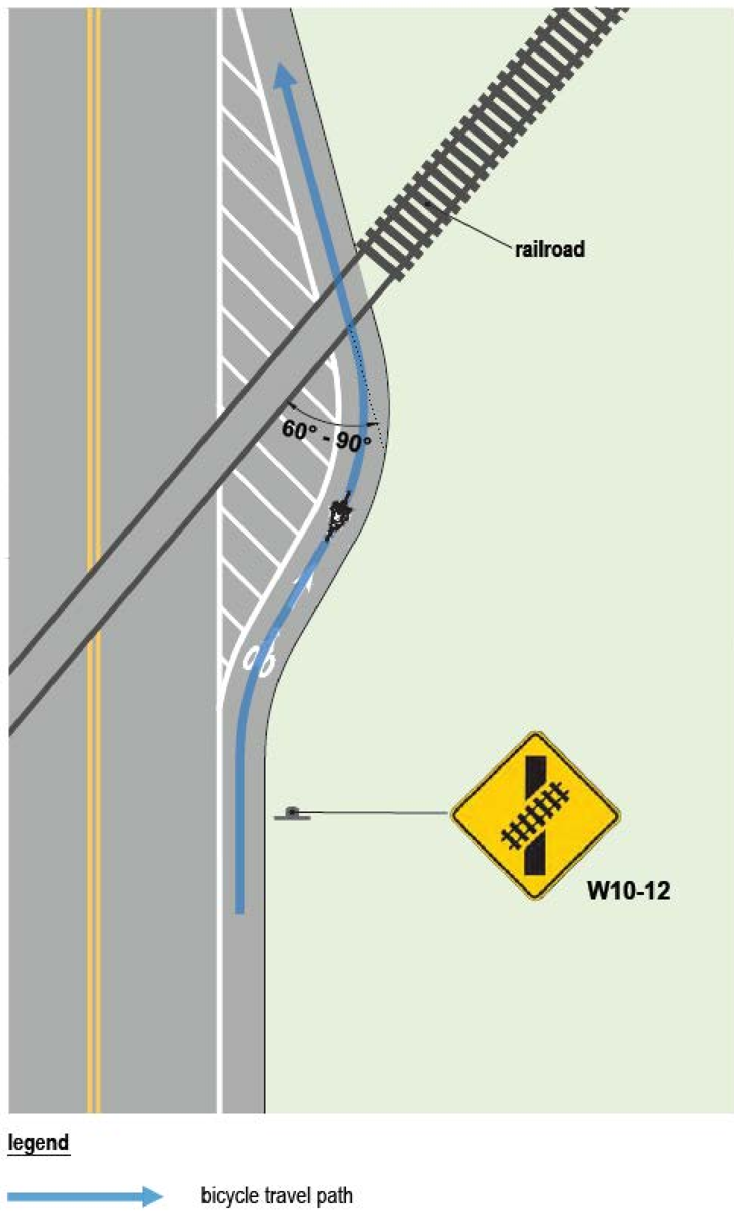


Figure 6-59. Railroad Crossing Example

(See Section 6.4.1.1 for additional signing and pavement marking guidance.)

6.4.6 Maintenance, Operations, and Work Zone

6.4.6.1 Maintenance Considerations

The large majority of bicycle crashes are due to falls and collisions due to surface defects and crashes with fixed objects located within bike operating spaces. Maintenance considerations should factor into the design approach for each bikeway and long-term maintenance programs should seek to proactively mitigate these issues to reduce crashes.

Seal coat projects must be properly swept for cyclists, including on the shoulders. Designers should also consider how paint markings affect cyclist safety over an interim period after a seal coat prior to placement of more permanent thermoplastic and prefabricated markings for cyclists.

Street sweepers and plows should be able to access the bikeway to minimize or eliminate the need for hand-sweeping to clear bike lane debris. If the bikeway clear space widths are not able to accommodate sweeping and snow removal equipment, the vertical elements should be mountable or designed so that the maintenance equipment can access the and clean the bike lane and that the vehicle and vertical elements are not damaged. Alternatively, investment in smaller equipment may be appropriate, or agreements with other agencies or organizations that have smaller equipment can be discussed. Designers should consider the following factors when planning for bikeway maintenance:

- Bikeways along drainageways should be swept promptly following large storm events;
- Low points on the bikeway should be kept at a minimum, or adequate drainage should be provided to keep stormwater flow outside the operating space of bicyclists during small storms;
- Repair from utility cuts should cover the entire width of the bike lane to prevent uneven riding surfaces;
- Vertical objects such as flexible delineator posts should be placed with at least 1-ft offset from adjacent travel lanes to reduce the frequency of replacement. Placement at intersections and driveways should carefully review vehicle turning movements; and
- Maintenance and operation crews should plan on replacing signs placed in the buffer zone, refreshing pavement markings, and trimming any adjacent vegetation on a regular basis.

6.4.6.2 Temporary Traffic Control for Bicyclists / Maintenance of Traffic

Construction projects often disrupt the public's mobility and access. Proper planning for bicyclists through and along work zones is as important as planning for motor vehicle traffic. The TMUTCD states that the needs and control of all road users (motorists, bicyclists, and pedestrians) through a temporary traffic control zone shall be an essential part of highway construction, utility work, maintenance operations, and the management of traffic incidents. Bicyclists should be expected on all roads unless prohibited (e.g. limited access highways), therefore work zone treatments such as temporary lane restrictions, detours, and other traffic control measures should be designed to accommodate bicyclists. Designers should incorporate the following recommendations into project construction plans:

- Maintenance of bicycle travel should be included whenever the need for temporary traffic control is being considered. Designers and construction personnel should determine how to maintain existing bikeways during construction. Options include accommodating bicyclists through the work zone or providing a suitable alternate route with the least amount of detour necessary. It is preferable for the alternate route to direct bicyclists to a bikeway that is equal to or lower in traffic stress than the existing route.
- Similar to other vehicular traffic, work zones should be compatible with bicycle travel. Work zone concerns for bicyclists may include road or path closures, sudden changes in elevation, construction equipment or materials, and other unexpected conditions. Accommodation in the work zone may result in the need for the construction of temporary facilities, including paved surfaces, structures, signs, and signals. The TMUTCD includes appropriate mode-specific detour guidelines in the section on temporary traffic controls. Where guidelines do not adequately cover a situation specific to bicycle use, designers should apply general vehicular guidelines and professional judgment.
- Work zone signs, construction vehicles, and other related construction materials should not be stored or placed within bikeways or on sidewalks that are open for use. Workers who routinely perform maintenance and construction operations should be aware of these considerations.
- For sections of separated bike lanes or shared use paths which are closed to bicyclists, advanced warning is necessary to allow bicyclists sufficient time and space to transition out of the bikeway. It is preferable for the transition to be kept as short as possible. This may require construction of temporary curb ramps to transition bicyclists to a street or sidewalk. It is also preferable to maintain physical separation from traffic where feasible, as separated bike lanes and shared use paths often attract people who are not comfortable operating in mixed traffic.

6.4.7. References

1. AASHTO Guide for the Development of Bicycle Facilities (2012, 4th Edition)
2. FHWA Bikeway Selection Guide (2019)
   https://safety.fhwa.dot.gov/ped_bike/tools_solve/docs/fhwasa18077.pdf
3. FHWA Bicycle and Pedestrian Facility Design Flexibility (2013 Memorandum)
   https://www.fhwa.dot.gov/environment/bicycle_pedestrian/guidance/design_flexibility.cfm
4. USDOT Policy Statement on Bicycle and Pedestrian Accommodation Regulations and Recommendations (2010)
   https://www.fhwa.dot.gov/environment/bicycle_pedestrian/guidance/policy_accom.cfm
5. TxDOT's Bicycle Tourism Trials Study (2018)
   https://www.txdot.gov/inside-txdot/modes-of-travel/bicycle/plan-design/tourism-study.html
6. FHWA Bikeway Selection Guide

7. National Center for Statistics and Analysis. Traffic Safety Facts 2014: A Compilation of Motor Vehicle Crash Data from the Fatality Analysis Reporting System and the General Estimates System. DOT HS 812 261. National Highway Traffic Safety Administration, U.S. Department of Transportation, Washington, DC, 2016.
8. FHWA. Shared Use Path Level of Service—A User's Guide. FHWA-HRT-05-138. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2006. https://www.fhwa.dot.gov/publications/research/safety/pedbike/05138/
9. 33 CFR 1.05-40.
10. FHWA Separated Bike Lane Planning and Design Guide (2015) https://www.fhwa.dot.gov/environment/bicycle_pedestrian/publications/separated_bikelane_pdg/separatedbikelane_pdg.pdf
11. Massachusetts DOT Separated Bike Lane Planning & Design Guide (2015) https://www.mass.gov/lists/separated-bike-lane-planning-design-guide
12. Torbic, D. J., K. M. Bauer, C. A. Fees, D. W. Harwood, R. V. Houten, J. LaPlante, and N. Rosebery. National Cooperative Highway Research Report 766: Recommended Bike lane Widths for Various Roadway Characteristics. NCHRP, Transportation Research Board, Washington, DC, 2014.

Additional Resources:

- Texas Manual on Uniform Traffic Control Devices (2011) http://ftp.dot.state.tx.us/pub/txdot-info/trf/tmutcd/2011-rev-2/revision-2.pdf
- NACTO Urban Bike Design Guide (2013) https://nacto.org/publication/urban-bikeway-design-guide/
- NACTO Designing for All Ages and Abilities (2017) https://nacto.org/wp-content/uploads/2017/12/NACTO_Designing-for-All-Ages-Abilities.pdf
- ITE Designing Walkable Urban Thoroughfares; A Context Sensitive Approach (2010) https://www.ite.org/pub/?id=E1CFF43C-2354-D714-51D9-D82B39D4DBAD
- FHWA Small Town and Rural Multimodal Network (2016) https://www.fhwa.dot.gov/environment/bicycle_pedestrian/publications/small_towns/

Chapter 7 — Miscellaneous Design Elements

Contents:

Section 1 — Longitudinal Barriers and Roadside Safety Hardware Criteria

Overview

This section discusses the features and design criteria for longitudinal barriers and roadside safety hardware including the following subsections:

- Concrete Barriers (Median and Roadside);
- Guardrail;
- Attenuators (Crash Cushions); and
- Roadside Safety Hardware Crash Criteria.

Concrete Barriers (Median and Roadside)

Application

Concrete barriers may be used to prevent the following:

- Unlawful turns;
- Out of control vehicles from entering the opposing traffic lanes;
- Unlawful crossing of medians by pedestrians; and
- Vehicles from encountering steep slopes or obstacles on the roadside.

Location

On controlled access highways, concrete barriers will generally be provided in medians of 30-ft or less. On non-controlled access highways, concrete barriers may be used on medians of 30-ft or less; however, care should be exercised in order to avoid the creation of an obstacle or restriction in sight distance at median openings or on horizontal curves. Generally, the use of concrete barriers on non-controlled access facilities should be restricted to areas with potential safety concerns such as railroad separations or through areas where median constriction occurs. Concrete barriers may be considered in medians wider than 30-ft based on an operational analysis or safety analysis.

Standard Installations

Medians for urban freeway sections generally are relatively narrow and flush. For new construction, an urban freeway usually includes a relatively narrow flush median (see Medians in Chapter 3 Section 2) with a concrete barrier.

In determining the type of barrier to be used, the primary consideration is safety, both for vehicular impacts and during maintenance activities. Field experience with concrete barriers indicates that, unlike guard fence systems, maintenance operations are not normally required following vehicular encroachment.

Reconstruction projects with median barriers should be considered on a project-by-project basis. Often, the structural capability of existing bridges may make the use of concrete median barriers infeasible due to increased dead load.

TxDOT's design standards and standard construction specifications provide more information on the design and construction details for concrete barriers. See Appendix A, Longitudinal Barriers for additional information on roadside safety hardware applications.

Guardrail

Application

Guardrail is considered a protective device for the traveling public and is used at points on the highway where a vehicle departure from the roadway would be a significant safety concern. Guardrail is designed to prevent impacts with roadside hazards by deflecting the vehicle so that it continues to move at a reduced velocity along the rail in the original direction of traffic. The limits of rail to be installed are shown on the plans; however, they may be adjusted in the field after the grading is completed to assure compliance with standard guardrail applications. See Appendix A for additional guardrail information.

Location

Guardrail should be installed in areas where the consequence of an errant vehicle leaving the roadway is judged to be more severe than impacting the guardrail. Guardrail should be offset at least 4-ft and desirably 5-ft or more, measured from the back of post to the nearest edge of roadside hazard. Refer to Appendix A, Longitudinal Barriers for additional information on offset guidance. At overpasses, guardrail should be anchored securely to the structure.

Standard Installations

Guardrail should be installed in accordance with current roadway standards.

End Treatments

Providing appropriate end treatments is one of the most important considerations in the design of guardrail. An untreated guardrail will stop a vehicle abruptly and can penetrate the passenger compartment. For more information on the installation of various types of end treatments, refer to Appendix A Section 5, End Treatment of Guard Fence and TxDOT's standard construction specifications and roadway standards.

Attenuators (Crash Cushions)

Application

Crash cushions or impact attenuators are protective devices that prevent errant vehicles from impacting fixed objects. This is accomplished by gradually decelerating a vehicle to a safe stop for head-on impacts or redirecting a vehicle away from the fixed object for side impacts.

Location

Attenuators are ideally suited for use at locations where fixed objects cannot be moved, relocated, or made breakaway, and cannot be adequately shielded by a longitudinal barrier. A common application of a crash cushion is in an exit ramp gore where a bridge rail end requires shielding. Crash cushions are also frequently used to shield bridge columns as well as roadside and median barrier terminals.

In temporary work zone applications using a parallel rigid barrier with narrow shoulder widths, if feasible, the rigid barrier should be tapered slightly on the approach to allow additional shy distance before the placement of the crash cushion. This will reduce incidental impacts. The taper rate of the rigid barrier, and alignment and placement of the crash cushion should conform to AASHTO Roadside Design Guide criteria, and the manufacturer's specifications for the specific crash cushion type.

Standard Installations

There are numerous types of attenuators that are in common use. When more than one system is under consideration, the designer should carefully evaluate the structural, safety, and maintenance characteristics of each candidate system. Characteristics to be considered include the following:

- Impact decelerations;
- Redirection capabilities;
- Anchorage and back-up structure requirements;
- Debris produced by impact; and
- Ease and cost of maintenance.

Crash Cushion Categories

Crash cushions are classified in one of three categories based on the reusability of the product after a head-on impact: sacrificial, reusable, and low maintenance.

Sacrificial

These units are typically filled with water or sand and typically require full replacement or substantial repairs either on-site or in a maintenance yard following an impact. These units have low initial costs and should be considered for sites that typically experience less than one impact every 18 months. Water-filled crash cushions are allowed for use in temporary work zones only.

Reusable

These units typically feature side fender panels, cartridges, or cylinders that absorb an errant vehicle's energy during impact. Typically, the cartridges, or cylinders and a nose piece will have to be replaced after an impact. These units should be considered in locations that typically experience impacts in the range between one impact every 18 months and less than 3 impacts per year.

Low Maintenance

These units typically utilize plastic cylinders, or hydraulic mechanisms to absorb energy. These require some maintenance after an impact to ensure proper performance during the next impact. These have high initial cost and should typically be considered at locations where they will experience 3 or more impacts per year.

For more detailed information on the installation of various types of attenuators, refer to TxDOT's standard specifications and roadway standards.

Roadside Safety Hardware Crash Criteria

AASHTO's *Manual for Assessing Safety Hardware* (MASH 2016), provides guidance for testing permanent and temporary highway safety features to assess safety performance of those features, replacing guidance defined in NCHRP Report 350. Guidance includes definitions of crash-test levels with specified vehicle, speed, and impact angle for each level.

Roadside hardware safety devices are categorized by Test Levels which define the impact conditions that the device is rated to withstand, based on structural adequacy, occupant risk, and vehicle trajectory. The standard MASH 2016 vehicles for testing categories include a small car (2,420-lbs.) and a large pick-up (5,000-lbs). TL-2 is used for low-speed roadways (45-mph or less); TL-3 is for high speed roadways (50-mph or greater); TL-4 includes the TL-3 criteria, plus additional testing for a delivery-type truck (22,000-lbs). The primary difference between MASH and the earlier NCHRP 350 criteria is the increase in size and height of the tested pick-up truck to account for the change in vehicle fleet, and to better simulate an SUV. Other changes include the small car weight, and angle of impact.

MASH Background

November 20, 2009, memorandum from David A. Nicol about AASHTO's *Manual for Assessing Safety Hardware* (MASH). This AASHTO manual supersedes NCHRP Report 350 for the purposes of evaluating new safety hardware such as longitudinal barriers, transitions, end terminals, crash cushions, breakaway/yielding supports, truck mounted attenuators, and work zone traffic control devices. It sets guidelines for crash testing and evaluation criteria for assessing test results. The joint AASHTO/FHWA implementation plan stated that all highway safety hardware accepted under the criteria in NCHRP Report 350 does not need to be retested to MASH criteria; may remain in place; and may continue to be manufactured and installed. However, all new hardware that is developed must be tested and evaluated according to MASH.

- May 21, 2012, memorandum from Tony Furst on the subject of Roadside Safety Hardware - Federal-Aid Reimbursement Eligibility Process and related Frequently Asked Questions. The memo establishes that States can certify that roadside safety hardware has been tested by an accredited crash test laboratory and meets MASH criteria, and can thus be eligible for reimbursement.
- January 7, 2016 memorandum from Thomas Everett on the subject of AASHTO/FHWA *Joint Implementation Agreement for Manual for Assessing Safety Hardware* (MASH). The memo discusses the agreement between AASHTO and FHWA that requires all new installations of safety hardware on the NHS to be evaluated using the 2016 edition of MASH.

Current TxDOT MASH Implementation Timetable/Policy

As product manufacturers and developers have tried to develop MASH 2016 compliant products, the FHWA in coordination with AASHTO has allowed additional flexibility with respect to the implementation of MASH 2016 compliant products by the respective states. The following are the categories of Roadside Safety Hardware Products and current TxDOT policy:

- W-Beam barriers and cast-in-place concrete barriers: Effective 12/31/2017, for all new permanent installations and full replacements, all W-Beam Barriers and cast-in-place concrete barriers shall be MASH 2016 compliant for projects let after this date.
- Guardrail End Treatments (SGTs): Effective Feb 28, 2018 all new permanent installations and full replacement SGTs must be MASH 2016 compliant regardless of project letting date.
- Cable barriers, cable barrier terminals, and crash cushions: In December 2019, the FHWA in collaboration with AASHTO provided updated guidance that allows the continued use of NCHRP 350 or MASH 2009 compliant devices for those categories of devices where a MASH 2016 alternative may not be available. The TxDOT Roadway Standards webpage provides standards (whether MASH 2016, MASH 2009, or NCHRP 350) that are available for use until further notice.
- Bridge rails, transitions, all other longitudinal barriers (including portable barriers installed permanently), all other terminals, sign supports, and all other breakaway devices: In December 2019, the FHWA in collaboration with AASHTO provided updated guidance that allows the continued use of NCHRP 350 or MASH 2009 compliant devices for categories of devices where a MASH 2016 alternative may not be available. Note that all current Bridge Railing Standards (BRG), Permanent Sign Support Standards (TRF), and Mailbox Standards (MNT) are MASH 2016 compliant.

- Temporary work zone devices (including portable barriers and water-filled crash cushions): Devices manufactured after 12/31/2019 are required to be MASH 2016 compliant. Such devices manufactured on or before 12/31/2019, and successfully tested to NCHRP 350 or MASH 2009, may continue to be used throughout their normal service lives. Note that certain temporary sign supports do not meet MASH 2016 criteria. Testing is on-going with these products. Also, trailer-type work zone devices such as arrow boards, and electronic portable message signs are not MASH 2016 compliant, but FHWA has currently exempted these devices due to safety benefits offered by their use.

All of the standards available on the respective TxDOT Division standards webpage are available for use until future notice. As additional MASH 2016 compliant items become available, the remaining NCHRP 350 or MASH 2009 items will migrate off the lists of available standards and the Districts will continue to be notified accordingly. A list of all available MASH compliant roadside safety hardware items and associated memoranda are available on the DES Division's (Roadway Design Section) Webpage.

Section 2 — Fencing

Overview

This section discusses the features and design criteria for fencing and includes the following subsections:

- Right-of-Way and
- Control of Access Fencing on Freeways

Right-of-way

Procedures for fencing highway right-of-way are in TxDOT's *ROW Acquisition Manual*.

Control of Access Fencing on Freeways

Control of access fence should be erected whenever it is necessary to prohibit unrestricted access to the through lanes by pedestrians, animals, and/or vehicles. The prohibition of access to the through lanes should be from private property, intercepted local roads, and unauthorized crossings from frontage roads to the through lanes. Table 7-1 describes the types of fences that should be used for various conditions.

Department standard designs should be used where applicable. Specially designed fences may be necessary in certain areas where sandstorms and snowstorms occur and for other special conditions.

Table 7-1: Use of Control of Access Fencing on Freeways

Condition	Type of Fence	Usual Location
Urban and suburban areas	Chain link fence of 4-ft usual height or 6-ft height where necessary for control of pedestrians	Variable[1]
Rural conditions where both large and small animals exist	Wire mesh fence with one or more strands of barbed wire or chain link, depending on the wildlife to be kept off the roadway	ROW line
Rural conditions where only large animals exist	Barbed wire fence with height of 4-ft or 5-ft	ROW line
Control of Vehicles	Post and cable fence with closely spaced posts	Variable[1]

Notes:

1. Where frontage roads are provided, control of access fence, when used, should be placed in the outer separation approximately equidistant between the mainlanes and frontage roads and at least 30-ft from the edge of mainlane pavement. Where the control of access line is at the right-of-way line, the control of access fence may be located at the right-of-way line and will serve a dual function as a right-of-way fence.

Section 3 — Pedestrian Facilities

This section discusses the features and design criteria for pedestrian facilities and includes the following subsections:

- 7.3.1 General
- 7.3.2 Elements of Design
- 7.3.3 Linear Pedestrian Facilities
- 7.3.4 Curb Ramp Design
- 7.3.5 Driveway Design Considerations
- 7.3.6. Intersections and Crossings
- 7.3.7 Overcrossings and Underpasses
- 7.3.8 Work Zone and Temporary Traffic Control Pedestrian Accommodations
- 7.3.9 Lighting
- 7.3.10 On-Street Parking
- 7.3.11 Transit Access
- 7.3.12 Railings Adjacent to Steep Slopes
- 7.3.13 Additional Considerations
- 7.3.14 Micromobility Vehicles

7.3.1 General

7.3.1.1 Purpose

The purpose of this Pedestrian Facilities section is to provide designers of roadways with tools and knowledge for planning and designing pedestrian facilities and other elements of the roadway that impact pedestrian safety and travel. These design decisions must serve people of all ages and abilities, including people too young to drive, who cannot drive, and people who choose not to drive.

For the purpose of this chapter, “pedestrians” are defined as people traveling by foot or using a mobility device, such as a wheelchair or motorized wheelchair, to assist in their travel. The range of design users and their differing characteristics are discussed in Section 7.3.2.1.

7.3.1.2 Design Imperative - Safety

Safety is a key factor in planning and design of roadway facilities for pedestrians as they are particularly vulnerable street users. Texas Administrative Code (TAC) Title 43 §15.122(2)(A) specifies that:

"Safety will be considered throughout the project development process. Each type of project will be evaluated, appropriate engineering studies will be completed, and appropriate design guidelines will be utilized with sound engineering judgment in order to accomplish the purpose of that particular transportation project. Safety is integral to properly engineering each project to address the anticipated needs and conditions."

The provisions of this chapter direct designers to consider geometric and operational features that will benefit pedestrians by reducing the likelihood of crashes and reducing the severity of crashes when they do occur.

As motor vehicle speeds increase, the risk of serious injury or fatality for pedestrians in the event of a crash also increases. (See Figure 7-1. Note that Figure 7-1 is for information purposes only based on the sources referenced. Actual stopping distances are based on many conditions such as roadway terrain, weather conditions, vehicle braking, tire design, and the human factor.) When pedestrians are expected, especially on high speed roadways, the separation of pedestrians from motor vehicle traffic, and the design of intersections that reduce motor vehicle speeds at conflict points and provide greater visibility of pedestrians must be considered. For the purpose of pedestrian guidance, the designated vehicular speeds will be the higher of the design speed or posted speed.

Designers should be aware of and incorporate findings from Road Safety Assessments being conducted by District staff and local jurisdictions for TxDOT facilities. These reports may contain infrastructure recommendations to improve roadway safety related to pedestrian travel that could drive the selection of specific aspects of project design.

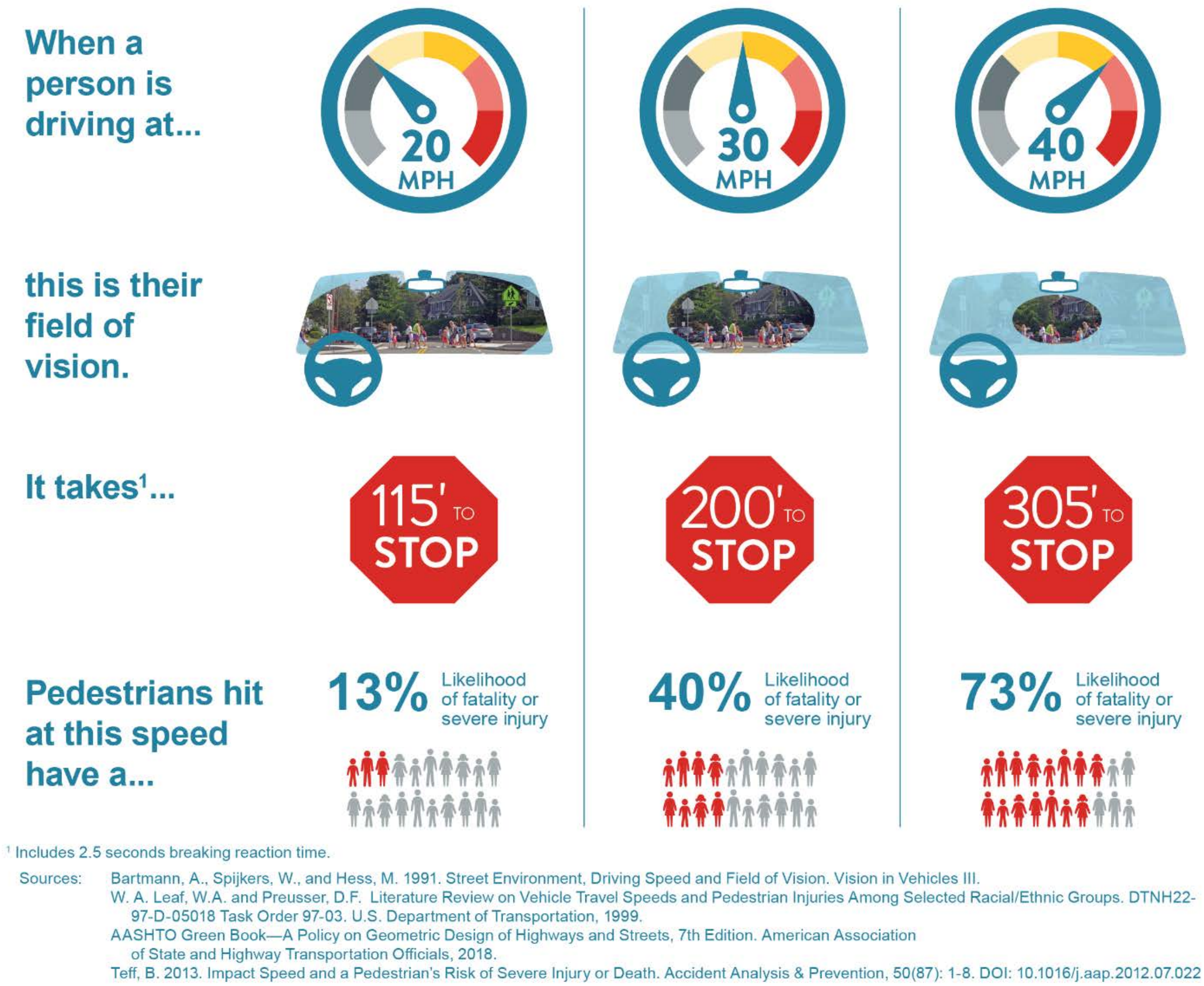


Figure 7-1. Speeds and the Risk of Serious Injury to Pedestrians.

7.3.1.3 Relationship to Other Policies, Laws, and Regulations

This design chapter is intended to support and comply with existing state and federal guidance. Under 23 U.S. Code 217(g)(1) it states, “Bicycle transportation facilities and pedestrian walkways must be considered, where appropriate, in conjunction with all new construction and reconstruction of transportation facilities, except where bicycle and pedestrian use are not permitted.”

Furthermore, this guidance is supported by FHWA’s *Bicycle and Pedestrian Planning, Program, and Project Development* guidance:

“Bicycle and pedestrian needs must be given “due consideration” under Federal surface transportation law (23 U.S.C. 217(g)(1)). This consideration should include, at a minimum, a presumption that bicyclists and pedestrians, including persons with disabilities, will be accommodated in the design of new and improved transportation facilities. In the planning, design, and operation of transportation facilities, bicyclists and pedestrians should be included as a matter of routine, and the decision to not accommodate them should be the exception rather than the rule.”

On March 11, 2010, a federal policy statement on Bicycle and Pedestrian Accommodations Regulations and Recommendations was signed by U.S. DOT. Recommended actions encourage the incorporation of "safe and convenient walking and bicycling facilities into transportation projects. Every transportation agency, including DOT, has the responsibility to improve conditions and opportunities for walking and bicycling and to integrate walking and bicycling into their transportation systems. Because of the numerous individual and community benefits that walking and bicycling provide — including health, safety, environmental, transportation, and quality of life — transportation agencies are encouraged to go beyond minimum standards to provide safe and convenient facilities for these modes."

Inclusion of pedestrian facilities in non-maintenance transportation projects is specified in the Texas Administrative Code Title 43, Part 1, §15.120-122. These sections identify design considerations for transportation projects that "involve the construction, reconstruction, rehabilitation, or resurfacing of a highway, other than a maintenance resurfacing project." These provisions apply to projects where TxDOT has design and construction or funding responsibilities. §15.122(1)(E) states "The department […] must consider the following factors when developing transportation projects: the access for other modes of transportation, including those that promote physically active communities." Further, §15.122(2)(D) states, "The factors provided in paragraph (1) of this section will be assessed when developing transportation projects. (D) Access for other modes of transportation will be considered during the project development process by developing plans and projects that contain, where appropriate, interconnections with other transportation facilities, including bicycle transportation facilities, pedestrian walkways, and trails."

7.3.1.3.1 Accessibility

Where pedestrian use is permitted, roadway designs and alterations must comply with accessibility requirements established by the Americans with Disabilities Act (ADA) Standards (2010) adopted by the U.S. Department of Justice (DOJ), the ADA Standards (2006) adopted by the U.S. Department of Transportation (DOT), Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-way (PROWAG), and the Texas Accessibility Standards (TAS). While the ADA Accessibility Guidelines (ADAAG) are the current enforceable federal regulations implementing the Americans with Disabilities Act (ADA), these pertain mostly to accessible design at public facilities and their grounds. Since the ADAAG does not adequately address features unique to public rights-of-way, the draft 2011 PROWAG was developed. The PROWAG has not been finalized by the U.S. Access Board nor adopted by the DOJ or the DOT so they are not enforceable standards. They do provide a useful framework to meet TxDOT's obligations to make our programs, services, and activities in the public right-of-way readily accessible and useable by all individuals including those with disabilities. The TAS provides some criteria on curb ramp design that echoes that included in PROWAG.

The TAS is issued by the Texas Department of Licensing and Regulation (TDLR) under the authority of Texas Government Code, Chapter 469. The standards are intended to be consistent to those contained in the DOJ ADA Standards (2010) and are generally the same except as noted. Both TAS and DOJ ADA Standards (2010) provide some criteria on curb ramp design that echoes that included in PROWAG.

As of May 15, 2017, changes to the Texas Administrative Code Title 16, Part 4, §68.102 allow the Texas Department of Licensing and Regulation (TDLR) to accept compliance with PROWAG in lieu of TAS for projects in the public right-of-way. Because the FHWA encourages the use of PROWAG as best practice, TxDOT designers must use PROWAG to achieve accessible design requirements in the public right-of-way. TAS and DOJ ADA Standards (2010) must be used for design and construction of buildings and sites such as TxDOT buildings, Safety Rest Areas, etc. A request for a design variance for any deviations from the applicable PROWAG, or if applicable, TAS requirements must be submitted to the TDLR for approval.

Throughout this section, PROWAG, Supplemental Notice of Proposed Rule Making (SNPRM), and other ADA-related guidance are generally referred to as “pedestrian accessibility guidelines,” except when the reference is to a specific document or regulation.

Further detail on the design implications of compliance are provided in Section 7.3.2.2.

7.3.1.3.2 Key Pedestrian Laws and Definitions

The Texas Transportation Code, Title 6 define laws associated with pedestrian facilities and pedestrian right of way (motorists, Chapter 545 and pedestrians, Chapter 552). The following is a summary of critical elements of these statutes which impact the design, operation, and use of pedestrian facilities on or crossing public roadways.

Marked and Unmarked Crosswalks (Texas Transportation Code §541.302)

Texas law defines a marked crosswalk as a pedestrian crossing that is designated by pavement markings. It also defines an unmarked crosswalk as the extension of a sidewalk or edge of the roadway where sidewalks are not provided across intersecting roadways. See Figure 7-2 below for location of unmarked crosswalk at an intersection. At midblock locations, unmarked crosswalks do not exist, and a crosswalk must be marked to legally establish the crosswalk. Crosswalks are marked to encourage pedestrian use of a crossing or to legally establish a crosswalk at midblock locations (see Figure 7-2).

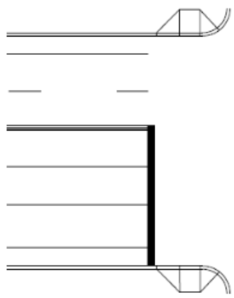


Figure 7-2. An Unmarked Crosswalk Exists at an Intersection as the Extension of the Sidewalk or Edge of Roadway Across the Intersecting Street, Regardless of the Presence of a Pedestrian Access Route.

Right of Way at Street Crossings

Motorists must stop and yield the right of way to pedestrians who are lawfully within crosswalks at:

All marked or unmarked unsignalized intersection crossings unless pedestrians are legally prohibited (Texas Transportation Code §552.003)

All marked or unmarked signalized intersection or midblock crossings where pedestrians enter the crossing on the WALK pedestrian phase or, in the absence of pedestrian signals, on the green phase for the parallel vehicular traffic (Texas Transportation Code §552.002)

In all locations, a pedestrian may not suddenly leave a curb or other place of safety and proceed into a crosswalk in the path of a vehicle so close that it is impossible for the vehicle operator to stop and yield.

Pedestrians must yield right of way to motorists at all unsignalized unmarked midblock locations and they are obligated to cross at marked crosswalks at an adjacent intersection if signals are in operation (Texas Transportation Code §552.005).

Use of Pedestrian Facilities

The law requires pedestrians walking along the roadway to walk on the left side of roadway facing traffic where sidewalks or other pedestrian facilities are not provided. Pedestrians are legally required to yield the right-of-way to all vehicles in these locations. Motorists must also exercise “due care” to avoid colliding with any pedestrian.

Where a sidewalk, or shared use path (sidepath) is present and accessible to pedestrians, the pedestrians are required to use those facilities. Due to the need of facilities designed for pedestrians to meet the respective ADA requirements (cross slope requirements, etc.), shoulders generally should not be used as pedestrian accessible routes. Other options should be pursued when designing pedestrian accessible routes.

7.3.1.4 Definitions:

- **Pedestrian Access Route (PAR).** A continuous and unobstructed path of travel provided for pedestrians with disabilities within or coinciding with a pedestrian circulation path.
- **Pedestrian Circulation Path.** A prepared exterior or interior surface provided for pedestrian travel in the public right-of-way.
- **Public Right-of-Way Accessibility Guidelines (PROWAG).** Guidelines from the US Access Board that address access to and on public streets, sidewalks, curb ramps, signals, on-street parking, crosswalks, and other public right-of-way components. See access-board.gov/prowag for more info.

7.3.1.5 Provision of Pedestrian Facilities

Based on TxDOT Administration guidance given for emphasizing pedestrian accommodations, pedestrian facilities must be considered for all types of transportation projects within urbanized settings. Therefore, the inclusion of these facilities must be considered when a project is scoped, with input from the local cities, metropolitan planning organizations, locally adopted bicycle and pedestrian plans, and the public, when applicable.

It is recommended to plan and design a project as if a pedestrian access route will be constructed, even if there is no need at the time of design. Extend culverts, acquire right-of-way, design intersections to allow for accessible crosswalks, and locating utilities and drainage systems to accommodate pedestrian systems. This will reduce the complexity and cost of future pedestrian infrastructure retrofit projects. It is difficult and costlier to retrofit pedestrian facilities when they were not considered in the initial design.

Site conditions sometimes make it difficult or impractical to meet certain accessibility requirements. In these situations, compliance is required to the extent practical in all other elements of design. Any non-compliant conditions must be documented with justification for non-compliance and must be included in the TxDOT ADA Transition Plan. For additional information on the TxDOT ADA Transition Plan contact the Landscape Architecture Section of the Design Division.

7.3.1.5.1 Sidewalks

Sidewalks, or a shared use path in cases where bicycle and pedestrian travel are intended to be accommodated together, must be provided in urbanized settings on:

- Full reconstruction projects;
- New construction projects;
- Projects within existing right-of-way that include pavement widening;

- Facilities that are part of a locally adopted sidewalk planning document;
- Facilities where there is evidence of pedestrian traffic:
  - Pedestrians are observed; or
  - There is evidence of a beaten path; or
  - There is significant potential for pedestrians to walk in the roadway.
- Facilities having existing pedestrian features;
- Facilities located on a route to school(s); or
- Facilities located on a transit route. All transit stops must be made accessible.

If it is determined that sidewalks will not be included in the project, then justification must be provided in the environmental document for not installing sidewalks. For the purposes of this guidance, "urbanized settings" include urban, urban core, suburban, and rural towns.

Although pedestrians are legally authorized to use the shoulder of the road for travel, it is preferable to provide accessible sidewalks in areas of known pedestrian activity or areas with increased development. A shoulder is not considered or designed to be a PAR.

7.3.1.6 Project Development

Planning for pedestrian and multimodal facilities must occur early and continuously throughout project development and must follow the TxDOT *Project Development Process Manual*. Early consideration of pedestrian facility design during the project development process is necessary to allow the active transportation network to be fully integrated into the overall transportation system.

The following items must be evaluated during project development:

- Sidewalks;
- Curb ramps;
- Driveway crossings;
- Vertical surface discontinuities not covered by routine maintenance operations;
- Accessibility compliance of existing traffic signal systems;
- Transit stops (through coordination with local transit authority);
- Railroad crossings;
- Installation of new, or upgrades to, accessible pedestrian signals and push buttons;
- Restriping existing crosswalk or other pavement markings;
- Adequate vertical clearance; and
- Other, as applicable

Considerations will differ between construction/reconstruction projects, and rehabilitation and resurfacing projects. Consideration of the following elements involves an assessment of whether the pedestrian facility and other roadway elements:

- Are compliant with ADA standards;
- Should be included in the project; and/or
- Should be upgraded or maintained as part of the project.
- Altered by the original scope of the project (per the FHWA/DOJ joint technical memo defining alterations and maintenance activities).

Further detail on specific elements is included below:

- Sidewalks

Suitability of width to context, while adhering to minimum width standards per accessibility guidelines (volume of expected users, see Section 7.3.3.2).

Suitability of placement to context (characteristics of adjacent roadway, see Section 7.3.3.1).

- Curb ramps

Provision of curb ramps during resurfacing projects.

Determination of the feasibility of filling gaps between existing ramps and nearby sidewalk segments that are disconnected from the street corner.

- Transit stops (through coordination with local transit authority)

Evaluate existing transit stop locations for crossing improvements.

Identification of isolated locations for potential relocation or to better serve with pedestrian route.

7.3.1.7 Maintenance and Operation

Project completion and ADA compliance of pedestrian facilities does not end at construction project closeout. Continuous maintenance is required to enable the pedestrian facilities to remain functional for every pedestrian. Maintenance of the pedestrian access route (PAR) includes, but is not limited to:

- Cleaning silt and debris that accumulate in ramps and on sidewalks;
- Removing vegetation overgrowth in the pedestrian zone;
- Repairing or replacing noncompliant surfaces due to settling, frost heaving, or root heave issues;
- Replacing fading crosswalks and pavement markings;

- Replacing worn detectable warning surfaces;
- Replacing missing or faded pedestrian signage;
- Maintaining 80” vertical clearance, 84” for signs, in pedestrian circulation path; and
- Adjusting pedestrian crossing signal timing with changes in traffic demands.

Pedestrian facilities should be incorporated into maintenance plans or agreements to maintain ADA compliance and general pedestrian safety and comfort.

7.3.2 Elements of Design

The following sections identify and describe the design user for pedestrian facilities and accessibility requirements. Principles specific to the design of linear pedestrian facilities, and crossings and intersections appear in their respective Sections 7.3.3.1 and 7.3.6.1.

7.3.2.1 Design User

The design user or “design vehicle” for pedestrian facilities must be a pedestrian relying on the use of a wheelchair, cane, crutches, or other equipment for use with navigating pedestrian facilities. If a design works well for people with disabilities, it generally works better for everyone. Designers should also consider the needs of pedestrians walking side by side, or using additional devices, such as a double-wide stroller.

7.3.2.1.1 User Characteristics

Designing a pedestrian facility must consider the basic principles of safety and comfort, as well as human factors like physical abilities, age, height, experience, and ability to perceive and react to potential conflicts. This focus on people’s varied abilities to travel as a pedestrian should be the basis for identifying the design user profiles that will inform key elements of design.

7.3.2.1.2 Mobility and Assistive Devices

Many pedestrians use varied types of mobility and assistive devices on a regular basis, while others may have temporary mobility disabilities that require them to use an assistive device. Some of the types of devices that are commonly used on pedestrian facilities are shown in Figure 7-3 below. These devices impact the spatial needs of pedestrians. These include, but are not limited to, the turning capabilities of wheeled mobility devices and the arc sweep of a cane used by someone with limited sight. Recommended dimensions and other specifications for pedestrian facilities in this guide generally meet the needs of these devices, and all meet the State and Federal requirements related to accommodating pedestrians with physical disabilities. See Section 7.3.2.2 for information on accessibility requirements.

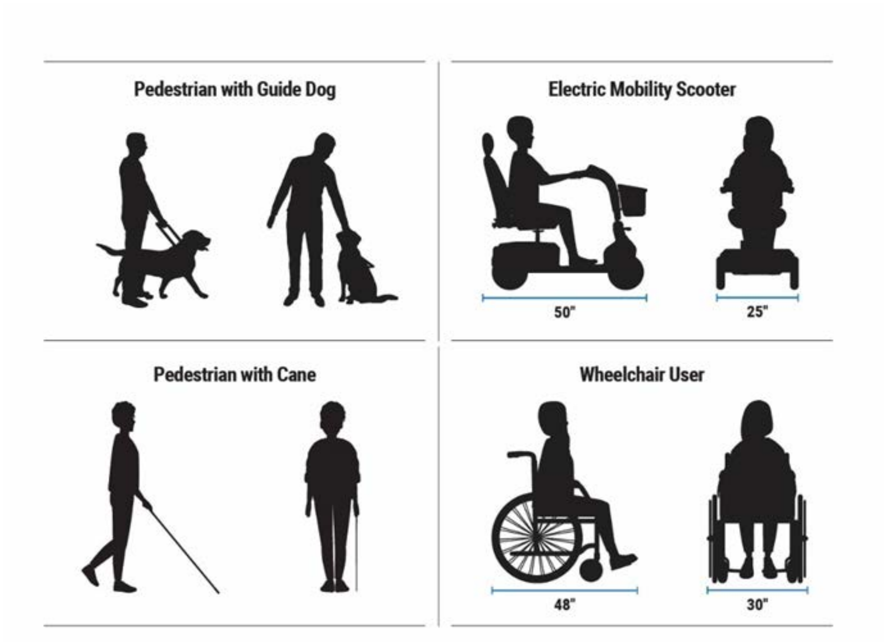


Figure 7-3. Mobility and Assistive Devices and Users.

7.3.2.1.3 Walking Speeds

Typical pedestrian walking speeds range from approximately 3 to 4-feet/s. Older people will generally walk at speeds in the lower end of this range. To accommodate most pedestrians, a walking speed of 3.5-feet/s must be used to determine signal timings, with a walking speed of 3-feet/s used where older pedestrians are expected or near schools. See the Highway Capacity Manual for criteria for slower walking speed consideration. See also the Texas MUTCD Part 4 Section 4.E06 with regard to pedestrian signal timing.

7.3.2.2 Accessibility Requirements

As described in Section 7.3.1.3.1 above, projects affecting pedestrian facilities must comply with accessibility requirements established in the PROWAG.

7.3.2.2.1 Pedestrian Access Routes

A pedestrian access route (PAR) is a continuous and unobstructed path of travel to accommodate pedestrians with disabilities within or coinciding with a pedestrian circulation path (a path provided for pedestrian travel). In the public right-of-way, a PAR must consist of one or more of the following components, all meeting PROWAG requirements as described in following sections:

- Sidewalks and other pedestrian circulation paths, or a portion of sidewalks and other pedestrian circulation paths, including across driveways;
- Pedestrian street crossings and at-grade rail crossings;
- Curb ramps and blended transitions;

- Pedestrian overpasses and underpasses, and similar structures, including any elevators used for their access;
- Ramps;
- Platform lifts (where elevators or ramps are not feasible); and
- Gates.

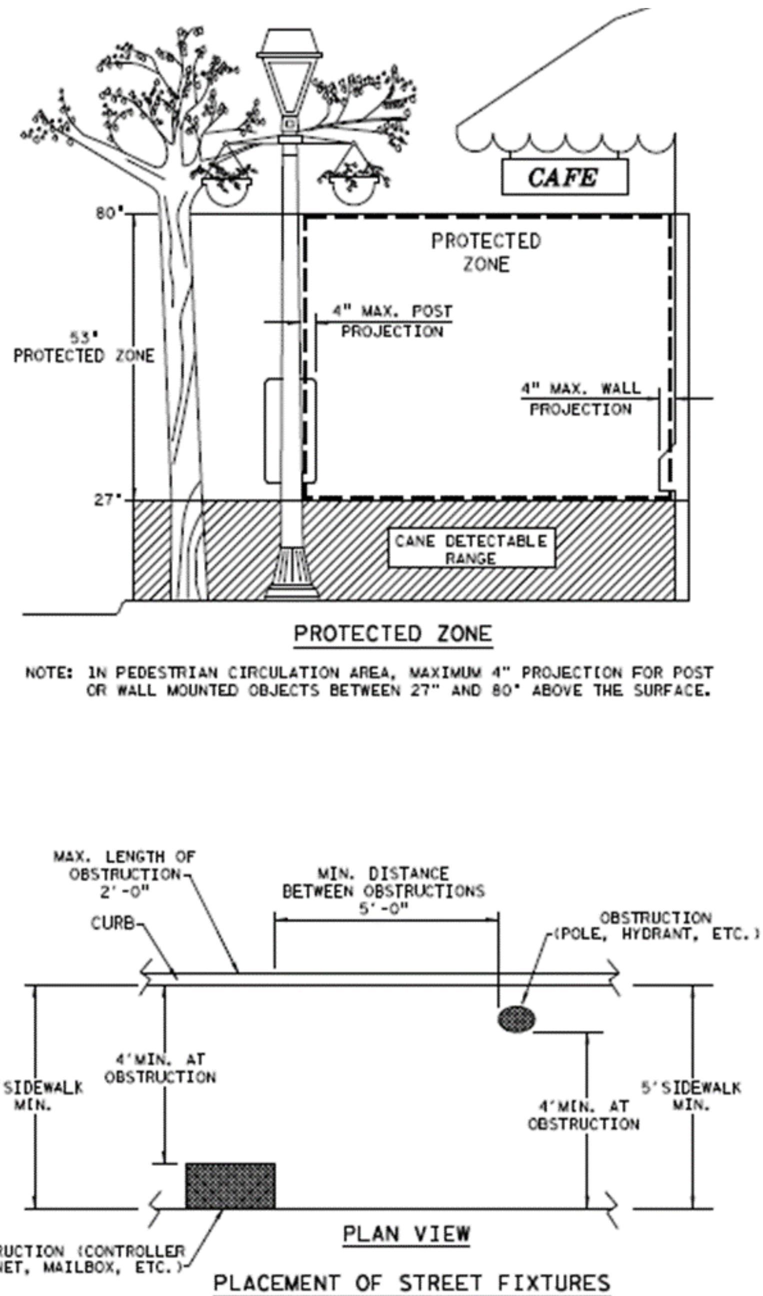


Figure 7-4. The Protected Zone (Top) and The Plan View (Bottom) Regarding Obstacles and Protruding Objects.

In most cases where pedestrian travel is expected (urbanized areas), the PAR along a roadway will be a sidewalk. Where a sidewalk is present, it must provide a PAR and follow PROWAG requirements stated below. In some cases where it is desirable to also accommodate bicycle travel, a shared use path (called a sidepath when along the roadway) will provide the PAR for pedestrians and be shared with bicyclists.

7.3.2.2.2 Clear Width

TxDOT's standard for the continuous clear width of a PAR is 5-ft minimum, exclusive of the width of the curb, which exceeds the ADA minimum of 4-ft. The clear width of a PAR may be reduced to 4-ft minimum for short distances, including across driveways, but passing spaces must be provided at intervals of 200-ft maximum. Passing spaces must be 5-ft minimum by 5-ft minimum. If a shared use path is provided, the full width of the shared use path must meet grade and cross slope requirements of a PAR. Width requirements for shared use paths are available in Chapter 6 Section 4, Bicycle Facilities.

7.3.2.2.3 Obstacles and Protruding Objects

Obstructions are objects along or overhanging any portion of the sidewalk or other pedestrian access route. These include street fixtures (signal and sign hardware, utilities, luminaires) and street furniture (benches, bus stops, drinking fountains, bicycle racks). The sidewalk may be reduced to a minimum of 4-ft around obstructions for a limited distance. Areas with a vertical clearance less than 80-in in height must have a barrier with the leading edge of the barrier at least 27-in in height. Pedestrian circulation paths should be designed with minimal obstructions. If obstructions cannot be removed or relocated, move or widen the pedestrian circulation path to maintain the continuous clear width.

Protruding objects are obstructions that protrude more than 4-in into the sidewalk or other pedestrian circulation path between 27-in and 80-in in height. Protruding objects below 27-in in height are considered detectable by cane. Protruding objects must be removed from the entire pedestrian circulation path or otherwise protected by a minimum 2.5-in-high foundation or curb to reduce the protrusion to the required 4 inch maximum.

For shared use paths, the SNPRM requires a minimum clearance of 8-ft for overhead obstructions, including those on pedestrian bridges.

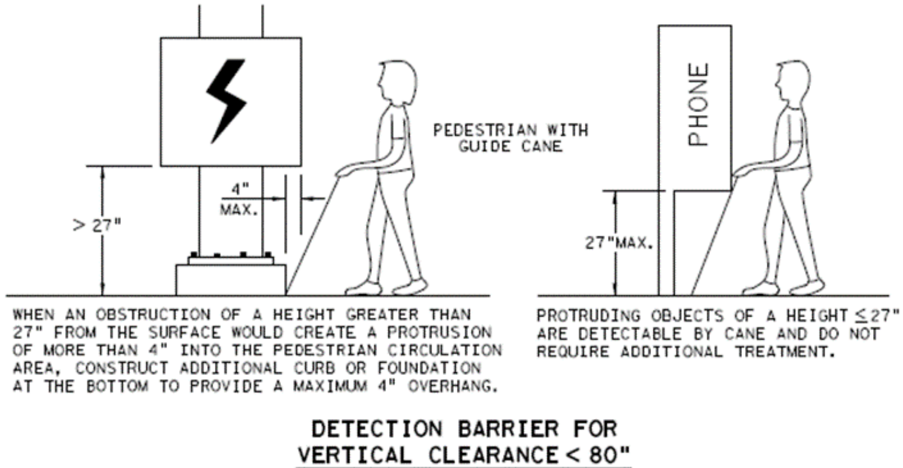


Figure 7-5. Detection Barriers for Vertical Clearance under 80".

7.3.2.2.4 Surface, Grade and Cross Slopes

Surface

The surface material of the PAR must be firm and stable and slip resistant. Concrete or asphalt surfaces are traditionally used, but other materials may be used provided they meet accessibility requirements. Avoid rough textures, such as cobblestone and heavily textured pavements as they can cause tripping hazards, confusion for pedestrians using tactile wayfinding cues, and painful vibrations for certain people using wheelchairs. Any gaps or openings must be small enough to prevent passing of a ½ inch diameter sphere. Vertical discontinuities must not be larger than ½ inch. Vertical discontinuities between ¼ inch and ½ inch must be beveled at a 2:1 slope.

Considerations for surfacing of shared use paths are discussed in Chapter 6 Section 4, Bicycle Facilities.

Detectable Warning Surfaces

Detectable warnings indicate to pedestrians, especially pedestrians with vision impairments, the presence of hazards in the line of travel, such as the edge of roadway and railroads, indicating that they should stop and determine the nature of the hazard before proceeding further. The contrasting color and truncated domes are visual and tactile queues for the pedestrians. Section R305 of PROWAG, Section 7.3.5.2.5 of this document, and the current TxDOT Pedestrian Facility Standards detail the application of detectable warnings in the public right-of-way.

Maximum PAR Grade (Within the Street or Highway ROW)

PAR running parallel (or adjacent) with roadway: For a PAR facility running parallel with a roadway with a steep general grade, the PAR grade must not exceed the general grade of the roadway.

PAR (independent alignment): For a PAR facility with an independent alignment, the grade of a PAR must not exceed 8.3 percent. A grade between 5 and 8.3 percent is considered a ramp and must meet ramp requirements.

Pedestrian Crossing: The running slope (measured parallel to direction of pedestrian travel) of a PAR has a maximum grade of 5 percent. Advisory R302.5 Grade (PROWAG) states that grade requirements must be applied to "sidewalks and other pedestrian circulation paths, pedestrian street crossings, and pedestrian overpasses and underpasses and similar structures."

https://www.access-board.gov/prowag/chapter-r3-technical-requirements/

Where compliance with the above is not practicable due to existing terrain or infrastructure, right-of-way availability, a notable natural feature, or similar existing physical constraints, a variance application must be submitted, and compliance is required to the extent practicable.

Cross Slope

The cross slope of a PAR must not exceed 2 percent. Due to construction tolerances, it is recommended that sidewalk cross slopes be specified in the plans as 1.5 percent to avoid exceeding the 2 percent limit when constructed. The cross slope requirements apply to PARs across bridge structures as well. The design of sidewalks at driveway crossings must also meet these cross-slope requirements.

Crossings at intersections with stop control (stop sign, signal, yield) must have a maximum 2 percent cross slope. Intersections without stop control or yield control must not exceed 5 percent cross slope. Where the PAR is contained within midblock pedestrian street crossings, the cross slope is permitted to equal the street or highway grade.

Accessible Pedestrian Signals

Section R209 of PROWAG specifies that where pedestrian signals are provided at pedestrian street crossings, they must include accessible pedestrian signals and pedestrian pushbuttons complying with sections 4E.09 through 4E.13 of the TMUTCD. This applies to new projects or those where the signal controller and software are altered, or the signal head is replaced (PROWAG Section R209.2). See Section 7.3.5.5.4 of this document for guidance and TRF Division Guidance.

7.3.3 Linear Pedestrian Facilities

Linear pedestrian facilities provide an area for pedestrian travel separated from automobile traffic. For pedestrian comfort and safety, particularly along roadways with high speed traffic, the goal is to maximize the width of the buffer between the sidewalk or shared use path and the adjacent roadway. As such, the desirable placement of the linear pedestrian facility is as close to the State ROW line as practical.

7.3.3.1 Characteristics Contributing to Pedestrian Comfort and Safety Along the Roadway

A variety of roadway characteristics impact pedestrian safety and the user experience for the pedestrians walking along the roadway. Note that the impact of these characteristics on crossings are addressed in subsequent sections.

- **Presence of a sidewalk.** Pedestrians are approximately 2.5x more likely to be involved in a crash when there is no sidewalk provided.
- **Speed of motor vehicle traffic.** Pedestrian fatalities rise exponentially as vehicular crash speeds increase. Pedestrians also experience a decrease in comfort when walking adjacent to high speed motor vehicle traffic. Pedestrian comfort and safety can be increased by increasing the width of the buffer zone and/or providing some type of vertical separation such as plantings. A row of parked vehicles between the sidewalk and the roadway also serves to improve pedestrian comfort.
- **Volume of motor vehicle traffic.** Higher motor vehicle volumes also decrease pedestrian comfort along the roadway, especially when operating at speeds above 20 mph. Similar options for separation as described above should be considered when designing sidewalks on higher volume roadways.
- **Vehicle type.** Similarly, a higher percentage of heavy vehicles on the adjacent roadway can decrease pedestrian comfort. Providing greater distance from the roadway or vertical elements may increase comfort.

7.3.3.2 Sidewalk Design on Curbed and Non-Curbed Roadways

Sidewalk design on curbed roadways includes designing three sidewalk zones: the pedestrian zone, the buffer furniture zone, and the frontage zone.

7.3.3.2.1 Pedestrian Zone

The pedestrian zone, also known as the “walking zone” or pedestrian circulation path. The Pedestrian Access Route (PAR), is the portion of the sidewalk dedicated to ADA accessible pedestrian movement.

The pedestrian zone should be wide enough to accommodate the volume and type of pedestrian traffic expected in the area. The minimum pedestrian zone width is 5-ft. In areas of higher expected pedestrian volumes, such as near schools, parks, in suburbanizing areas, and along transit routes, a pedestrian zone of 6-ft or more is desired in order to provide for side-by-side walking and passing. Pedestrian zone widths of 8-ft or more may be appropriate in downtown commercial areas and in other areas with concentrated pedestrian traffic.

Where necessary to avoid an obstruction or to cross a driveway while maintaining the maximum 2 percent cross slope, the pedestrian zone may be reduced to 4-ft for short distances. Sidewalks less than 5-ft in width must include passing sections of 5-ft x 5-ft no more than every 200-ft.

While sidewalks do not need to be perfectly straight, the pedestrian zone should not weave back and forth in the right-of-way for no other reason than to introduce curves. Meandering sidewalks create navigational difficulties for pedestrian with vision impairments

Curbed Roadways

Where a sidewalk is placed immediately adjacent to the back of curb with no buffer, a minimum pedestrian zone width of 6-ft is desirable and 7-ft preferred to allow additional space for street and highway hardware and allow for the proximity of moving traffic. Where the curb-adjacent sidewalk is in a commercial district, a minimum of 8-ft is preferred to accommodate pedestrian volumes and passenger access from parked vehicles on street.

Uncurbed Roadways

The same width requirements for the pedestrian zone apply to sidewalks on uncurbed roadways. In these cases, sidewalk should be placed between the ditch and the right-of-way line, not between the ditch and edge of the vehicular travelway.

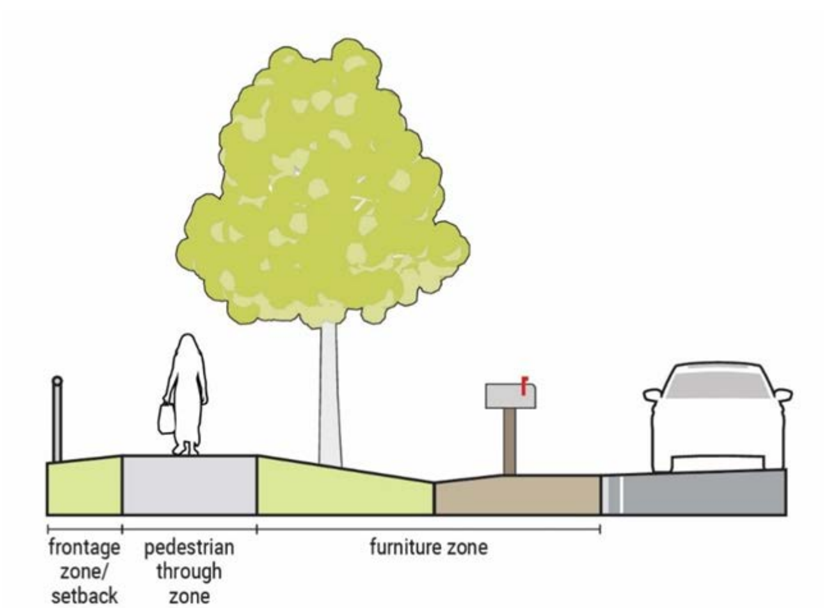


Figure 7-6. An Uncurbed Roadway.

7.3.3.2.2 Buffer (Furniture) Zone

For pedestrian comfort, especially adjacent to high-speed traffic, it is desirable to provide a buffer (commonly called a furniture zone in urbanized areas), between the vehicular travelway and the pedestrian zone.

Curbed Roadways

On curbed roadways, the buffer is the space between face of curb and the edge of the pedestrian zone. Pedestrian zone placement immediately adjacent to the curb should be avoided where at all possible. Buffers provide space for other operational needs in the right-of-way, including signposts, bus stops, parking meters, utility connections, and light poles. They also provide space for plantings and other landscape development which enhance pedestrian comfort through providing shade and a greater sense of buffering and protection from adjacent traffic. See the Landscape & Aesthetic Design Manual for appropriate plant selection and application. Buffers can also allow the pedestrian zone to continue at a constant grade across driveways as described in Section 7.3.4.1.2 and provide for much better curb ramp designs at intersections.

While wider clear zones are appropriate for freeways and high-speed roadways, the AASHTO Roadside Design Guide recognizes that there are practical limitations to clear zones on low-speed curbed streets. See Table 2-12 and Appendix A, Section 2 for clear zone guidance. In urban, urban core, suburban, and small-town rural settings where pedestrian activity is expected, traffic speed is generally lower, and, depending on the context, roadway design may incorporate street furniture and/or plantings to create a sense of enclosure. This provides a traffic calming effect, which may increase comfort and safety for vulnerable road users

For curb and gutter sections, the buffer zone should be at least 4-ft wide measured from face of curb, although more buffer space is preferable. For roadways with higher speed traffic, particularly those with speeds over 35mph, designs should consider providing additional buffer width to protect pedestrians from vehicle traffic and provide a more comfortable walking environment. The respective desirable clear zones from Table 2-12 would be the desirable buffer width.

Uncurbed Roadways

On uncurbed roadways, the buffer zone is defined as the space between the edge of the vehicular travelway pavement and the edge of pedestrian zone sidewalk pavement. The minimum buffer zone is 10-ft for both low- and high-speed conditions. The desirable buffer would be the respective desirable clear zone values from Table 2-12 in Chapter 2.

This space is often the location of drainage ditches. If the ditch has a backslope of 3:1 or steeper, a separation of at least 2-ft should be provided between the top of slope and the edge of pedestrian zone pavement.

At driveways, bus stops, mid-block crossings, and anywhere else where a pedestrian or bicyclist would need to access the sidewalk/sidepath, an at-grade crossing of the ditch is required and may be provided through use of a sidewalk bridge, culvert, trench drain, or other means.

7.3.3.2.3 Frontage Zone

The frontage zone is the area between the pedestrian zone and the property line, typically applicable in areas with buildings directly adjacent to the ROW line.

Curbed Roadways

Frontage zone width may vary between 1- and 2.5-ft depending on building setback requirements and adjacent land uses. A 1-ft frontage zone (resulting in the pedestrian zone being 1-ft off the property/ROW line) provides room for maintenance space; with the 1-ft space, a construction easement will not be necessary if sidewalk maintenance needs to occur. In downtown areas, wider frontage zones may be considered for accommodation of commercial uses such as product displays and outdoor dining seating.

Non-Curbed Roadways

Typical land use context for non-curbed roadways would not necessitate a frontage zone other than for providing maintenance space. Setbacks may effectively function as frontage zones in these areas, separating the paved pedestrian zone from adjacent structures on private property.

7.3.3.3 Shared Use Paths

A shared use path is defined as a multi-use path designed for use by bicyclists and pedestrians, including pedestrians with disabilities. Shared use paths are physically separated from motor vehicle traffic by an open space or barrier and are either within the highway right-of-way or within an independent right-of-way.

Shared use paths are used by pedestrians and must meet the accessibility requirements of the Americans with Disabilities Act (ADA). Paths in the public right-of-way that follow the roadway and function as sidewalks, commonly referred to as sidepaths, should be designed in accordance with the proposed PROWAG, or subsequent guidance that may supersede PROWAG in the future. Shared use paths built in independent right-of-way should meet the draft accessibility guidelines in the Supplemental Notice of Proposed Rulemaking (SNPRM) on Shared Use Paths or any subsequent rulemaking that supersedes the SNPRM. Where pedestrian and bicycle travel are accommodated on separate pathways, only the pedestrian pathway must meet the requirements of the above guidelines.

7.3.3.3.1 Cross Slope

See Section 7.3.2.2.2 for information regarding the accessibility requirements related to cross slope.

7.3.3.3.2 Grade

See Section 7.3.2.2.2 for information regarding the accessibility requirements related to grade.

If the path is at or approaching 5 percent for a significant length (1000-ft or more), this sustained grade would be quite noticeable for a bicyclists or wheelchair user and maximum grade of 3 to 4 percent should be considered. Alternately or in addition to a lower sustained grade, level-graded pull-outs could be provided at strategic locations for users to get out of the travel path to pause or rest, or resting intervals of flatter grades may be provided. Designers may also consider the provision of accessible pedestrian handrails located at the edge of the path to assist pedestrians in traversing steeper grades. See Section 7.3.13 for guidance regarding protective rails adjacent to steep slopes.

Provision of a handrail may impact shared use path width; see Chapter 6 Section 4, Bicycle Facilities for information regarding the impact of handrails on path width.

7.3.3.3.3 Additional Requirements

See the above guidance documents of PROWAG and SNPRM for additional specific requirements for shared use paths that are intended to serve as pedestrian facilities.

More information on shared use paths can also be found in Chapter 6 Section 4, Bicycle Facilities.

7.3.4 Curb Ramp Design

Curb ramps are a fundamental element of the PAR because they form the vital connections between sidewalks and street crossings. Curb ramps are required at all pedestrian crossings, including mid-block crossings, unless the crossing is brought to the level of the sidewalk. The TxDOT preferred standard is two curb ramps per corner, each aligning with desired paths of travel (see Section 7.3.5.2.6). All curb ramps must be compliant with PROWAG. Refer to the current TxDOT Pedestrian Facility Standards for further detail on curb ramp design.

7.3.4.1 Curb Ramp Locations

Curb ramps must be installed to connect the pedestrian access routes at each pedestrian street crossing. A pedestrian street crossing is considered to be present if:

- There is a sidewalk or shared use path crossing a curb,
- There are pedestrian signal heads or detection buttons indicating pedestrian presence,*
- There is a marked crosswalk, or
- There is a school crossing.

*The ADA does not require installation of curb ramps in the absence of a pedestrian walkway with a prepared surface (even when items 3 and 4 on the list above exist).

Curb ramp placement may be affected by the location and placement of streetscape elements and utilities, including inlets. The location of fixed objects (e.g., poles, signal cabinets, etc.) should not limit access for pedestrians and bicyclists using sidewalks and curb ramps. Curb ramps and crosswalks should be designed to drain water away from curb ramps, reducing risk of pooling (and icing) across ramps. The approach to curb ramp landings should minimize the number of movements needed to reach the curb ramp. An excessive number of movements may be hard for impaired pedestrians to detect and maneuver. See Section 7.3.5.1.2 for the design of curb ramps at driveways.

7.3.4.2 Design Considerations

Manhole covers, grates, and obstructions should not be located within the curb ramp, maneuvering area, landings or turning spaces. Curb ramps should not be located where pedestrians must cross drainage structures such as inlets and manholes. In new construction, curb ramp design must consider both the location of the curb ramp and the drainage structures. Additional drainage inlets should be located on the upstream side of all curb ramps where practical in order to avoid running water across the foot of the ramp. Curb ramps must be located away from low points of the curb return. For more information on drainage design, see Chapter 2 Section 7.

7.3.4.3 Curb Ramp Components

The basic components to the standard curb ramp design are the following (see Section 7.3.4.4, Design Elements, for specifications on each element):

- Ramp – The ramp is the portion of the curb ramp that pedestrians traverse, delivering them from the crosswalk to the sidewalk.
- Turning Space (Landing) – Turning spaces provide an area for mobility device users to maneuver into or out of the curb ramp, or to simply bypass it. Turning spaces are required at the top or bottom of each ramp and have a maximum slope of 2 percent in any direction.
- Flares – Flares are graded transitions from a curb ramp to the surrounding sidewalk. The flares of a curb ramp are not intended to be used by pedestrians using mobility devices. Flares may be seen as a signal to identify the presence of a curb ramp. Flares are required when the pedestrian circulation path crosses the curb ramp, or when the curb ramp is adjacent to a walkable surface.
- Gutter – The gutter is the curb and gutter section within the ramp width limits that connects the roadway travelway to the ramp.

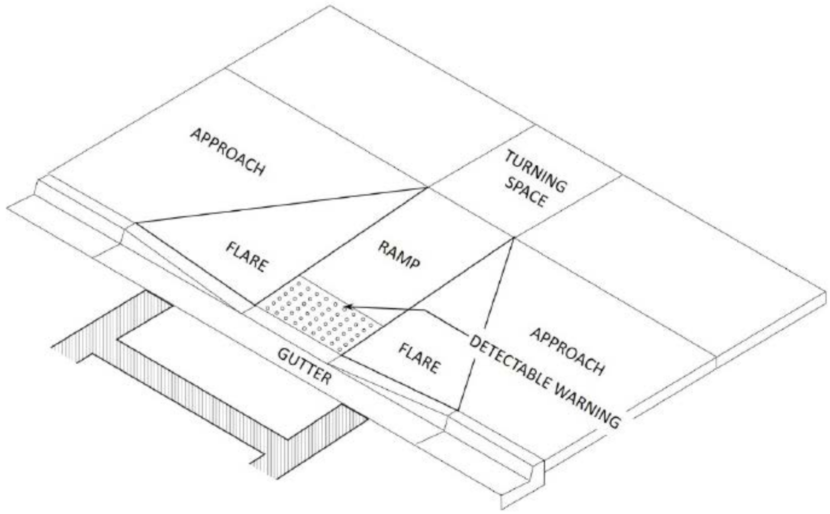


Figure 7-7. Curb Ramp Components.

7.3.4.4 Design Elements

7.3.4.4.1 Grade

The grade of a curb ramp (also referred to as the running slope) is the grade measured along the direction of pedestrian travel. The grade of the curb ramp should be a maximum of 8.3 percent (1V:12H). Where the transitioning slope between the sidewalk and roadway is less than 5 percent, it is considered a blended transition.

The grade, in any direction, of flares measured along the back of curb must not exceed 10 percent measured parallel to the curb.

Transitions from the curb ramp to gutter and/or road surfaces must be flush (level) and free of abrupt surface changes. Gutters and/or road surfaces immediately adjacent to curb ramps must have a maximum counterslope of 5 percent toward the curb ramp. However, a rapid change in grade at the base of the curb ramp and gutter may be difficult for wheelchair users if the footrests or anti-tip wheel cannot clear the ground surface. Therefore, the maximum recommended change in grade between a curb ramp and gutter (or road surface) is limited to 11 percent. If the algebraic difference in grade exceeds 11 percent, a 24 in level strip should be provided to transition from the curb ramp to the gutter (or road surface) to avoid tripping concerns (see Figure 7-8).

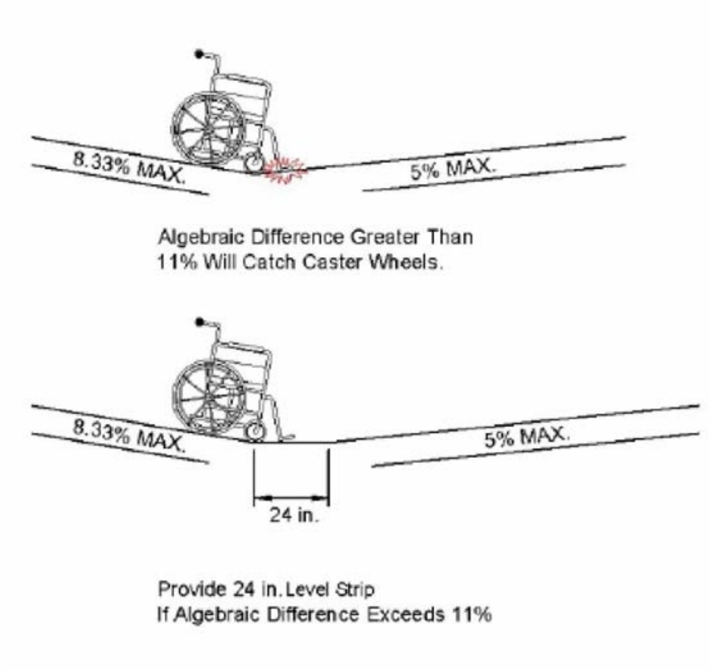


Figure 7-8. Grade Considerations for Curb Ramps.

7.3.4.4.2 Cross Slope

The curb ramp cross slope is the slope perpendicular to (i.e., across) the direction of pedestrian travel on the curb ramp. The cross slope of the curb ramp should not exceed 2 percent. However at midblock pedestrian street crossings and intersection crossings without stop or yield control, PROWAG will permit curb ramp cross slopes equal to the street or highway grade. It is desirable to keep the cross slope value low because people with mobility disabilities often have difficulty negotiating a grade and cross slope simultaneously. Since the grade of the curb ramp will usually be significant, the cross slope should be minimized; also, the transition from the cross slope on the ramp to the steeper street grade should not be sharp so as to cause unstable conditions for wheelchairs. Transitioning the cross slope of the curb ramp to match the roadway profile behind the curb allows the pedestrian to adjust to the road profile in the safety of the space behind the curb. Transitioning behind the curb also keeps stormwater flow lines intact.

7.3.4.4.3 Width

It is recommended that the curb ramp be the full width of the sidewalk or approaching shared use path. The minimum width of a curb ramp is 4-ft, exclusive of flared sides. Some ramp types may require a minimum width of 5-ft. Refer to the latest TxDOT Pedestrian Facility Standards for ramp types.

7.3.4.4.4 Turning Spaces (Landings)

A turning space is a landing area that allows users to maneuver on and off the curb ramp. Turning spaces may overlap the path of travel for pedestrians who are continuing along the sidewalk and do not want to cross the street. The maximum slope for a turning space is 2 percent in all directions. The minimum dimensions of a turning space are 5-ft x 5-ft. They should be provided at either the top or bottom of the curb ramp depending on curb ramp type or design, and/or whenever a turning maneuver is needed for pedestrians to maintain their orientation to the direction of the ramp.

7.3.4.4.5 Detectable Warnings

Curb ramps must contain a detectable warning surface that consists of raised truncated domes that comply with PROWAG. Detectable warning surfaces must be used at the following locations on the pedestrian access route:

- Curb ramps and blended transitions at pedestrian street crossings
- Pedestrian refuge islands
- Pedestrian at-grade rail crossings not located within a street or highway
- Boarding platforms at transit stops for buses and rail vehicles where the edges of the boarding platform are not protected by screens or guards, and
- Where the side of the boarding and alighting areas facing the rail vehicles is not protected by screens or guards

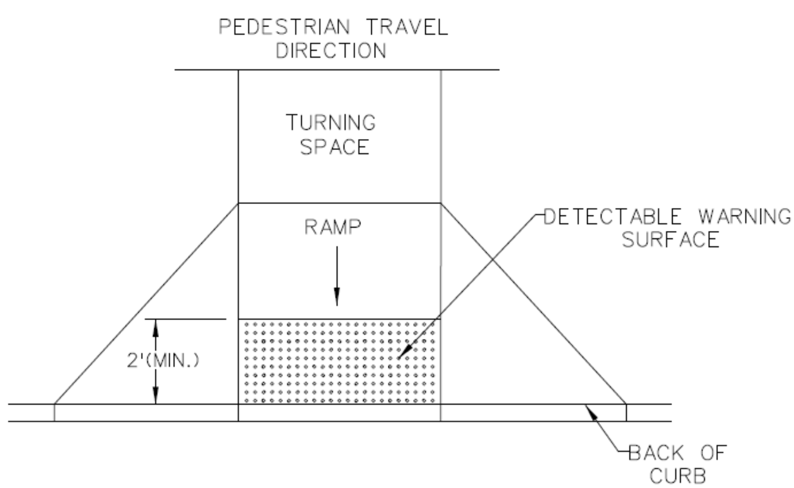


Figure 7-9. Detectable Warning Strips with a Minimum of 2-ft.

Detectable warnings must contrast visually with the adjacent surfaces, either dark on light or light on dark. Detectable warning surfaces must extend a minimum of 2 ft in the direction of pedestrian travel and must extend the full width of the curb ramp or blended transition. Detectable warning surfaces must be placed at the back of the curb or edge of street; detectable warning surfaces may be curved along the corner radius.

Detectable warning materials must meet current TxDOT Pedestrian Facility Standards materials specifications.

7.3.4.4.6 Side Treatments

Side treatments of curb ramps include flares and returned curbs. See images below for an example of flares and returned curbs.

Flares

Flares must be provided for curb ramp designs where the pedestrian circulation path crosses the curb ramp. Flared sides should be sloped at 10 percent maximum, measured parallel to the curb.

Returned Curbs

Returned curbs do not use flares, but instead curbs on the sides of the curb ramp. Returned curbs may be used only where pedestrians would not normally walk across the ramp, either because the adjacent surface is planted, substantially obstructed, or otherwise not meant for pedestrian travel.

Figure 7-10. Curb Ramp with Flares (left) and Returned Curb (right).

7.3.4.5 Curb Ramp Types

Curb ramps are classified as perpendicular, directional, parallel, combination or blended transition per TxDOT PED Standards.

Diagonal curb ramps, defined as one ramp serving two crossing directions, are no longer permissible and may only be used where existing physical constraints prevent two curb ramps from being constructed.

7.3.4.5.1 Perpendicular

Perpendicular curb ramps are the preferred ramps for application over parallel and directional curb ramps, with an adequate landing provided for pedestrians for each approach and the alignment of travel to traffic is generally perpendicular. Where practical, the curb ramp path should be aligned with the crosswalk. At large curb return radii, it may not be possible to provide a curb ramp that is both aligned with the crosswalk and exactly perpendicular to the curb face.

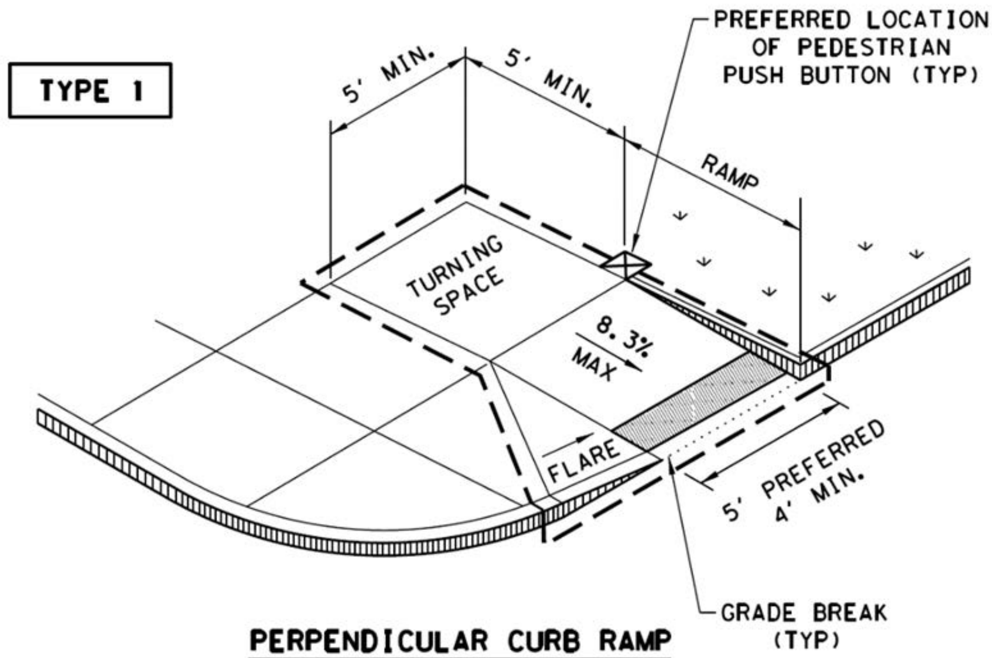


Figure 7-11. Specifications for a Perpendicular Curb Ramp.

7.3.4.5.2 Parallel

Parallel curb ramps have two ramps leading down towards a center level landing at the bottom between both ramps. Since pedestrians must make a turning movement at the bottom of the ramp to align themselves with the street, it is crucial to provide an adequate turning space at the base of the ramp. Parallel ramps are one solution on narrow sidewalks where perpendicular ramps are not feasible. Parallel curb ramps typically include a curb wall at the back of the ramp.

Figure 7-12. Parallel Curb Ramp.

7.3.4.5.3 Additional Sub-Types

Other types of curb ramps include combination curb ramps blended transitions, and ramps at median islands. They are facilities that combine design elements of both perpendicular and parallel curb ramps. Slopes, widths, and other conditions must be consistent with common curb ramp requirements. Combination curb ramps are useful in areas of limited right of way and areas with high sidewalks. Note, the secondary ramp is considered part of the combination curb ramp and may not meet sidewalk ramp requirements.

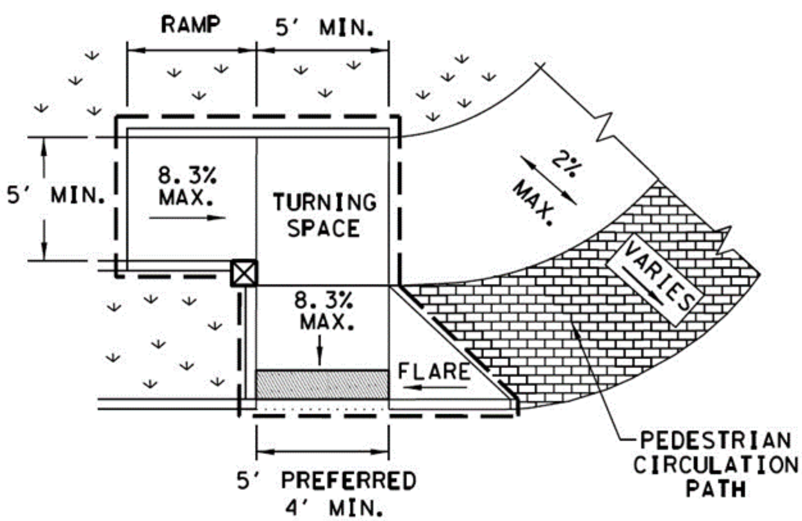


Figure 7-13. Combination Curb Ramp. Key Ramp Slopes and Turning Space are Maintained.

Blended Transitions

With a blended transition, the sidewalk elevation is lowered to the street level with a gradual change in slope. The maximum grade in the direction of pedestrian travel is 5 percent, provided there is a pedestrian access route at the top of the slope. Otherwise, the maximum slope is 2 percent. Blended transitions without accessible pedestrian signals (APSs) should be used sparingly since they provide limited directionality for pedestrians with vision disabilities.

Figure 7-14. A Blended Transition Curb Ramp.

Curb Ramps at Median and Channelization Islands

Ramps at raised median islands must have a minimum 5-ft x 5-ft landing between curb ramps. Where it is difficult to provide a 5-ft x 5-ft landing because of space restrictions, consideration can be given to only partially raising the landing above street grade, which allows for shorter curb ramp lengths and a longer landing. Another alternative is to provide a level cut-through. If the width of the median is 6-ft or greater, tactile detectable warnings are still required though no curb ramp is present. For medians less than 6-ft, pedestrian refuge is not available and detectable warnings should not be utilized. These designs are also applicable for channelization islands.

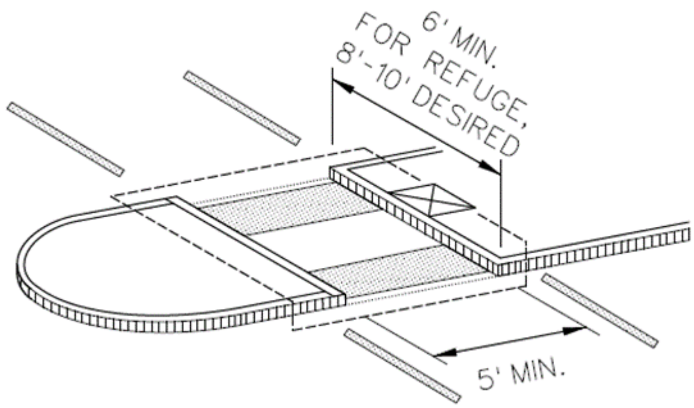


Figure 7-15. Curb Ramp with Level Cut-through at Median.

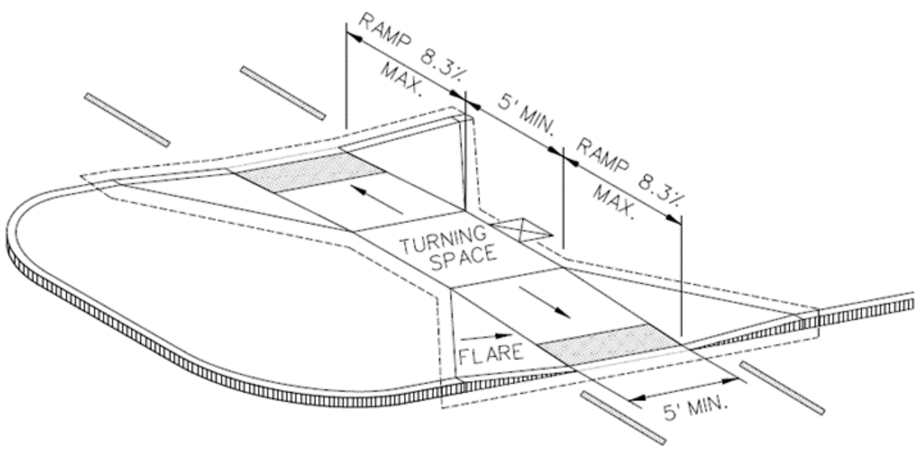


Figure 7-16. Curb Ramps at a Median.

7.3.4.6 Curb Ramp Evaluation

Curb ramps that meet ADA standards do not need replacement. However, curb ramps that do not meet ADA standards, must undergo evaluation to determine whether and how they will need to be replaced or retrofitted.

Criteria for installing or updating curb ramps include:

- New construction;
- New projects that are considered an alteration;
- New projects that alter existing sidewalks or prepared surfaces with barriers;
- Existing curb ramps that do not meet ADA standards in place at the time that the curb ramps were constructed;

- Resurfacing is an alteration that triggers the requirement to add curb ramps or bring existing curb ramps into compliance with current standards if it involves work on a street or roadway spanning from one intersection to another and includes overlays of additional material to the road surface, with or without milling. Examples include, but are not limited to the following treatments or their equivalents:
  - Addition of a new layer of asphalt;
  - Reconstruction;
  - Concrete pavement rehabilitation and reconstruction;
  - Mill and fill or mill and overlays;
  - Micro-surfacing and thin lift overlays; or
  - Cape seals and in-place asphalt recycling.

Maintenance projects do not trigger an evaluation of existing curb ramps and the addition of new curb ramps. Examples of maintenance activities include, but are not limited to the following treatments or their equivalents:

- Seal coats (chip seals);
- Fog seals and slurry seals;
- Joint repairs;
- Diamond grinding;
- Pavement patching; or
- Spot repairs.

7.3.4.7 Design Variance to Curb Ramp Replacement

Where existing physical constraints make it impracticable for altered elements, spaces, or facilities to fully comply with the requirements for new construction, compliance with the ADA is required to the extent practicable within the scope of the project. Variance application must be submitted for any project that does not achieve full compliance.

7.3.5 Driveway Design Considerations

Like intersections, driveways are locations of potential conflict between pedestrians and vehicles. As such, they should be designed to maximize visibility between motorists and pedestrians, slow motor vehicle turning speeds, and provide adequate space for vehicles to queue while stopping and yielding to pedestrians in the driveway crossing. Driveways should be consolidated and/or minimized on streets with anticipated high pedestrian volumes, those with a high density of pedestrian destinations, and commercial streets.

Designers refer to Appendix C for more detailed driveway design guidance.

7.3.5.1 General Guidelines

Accommodating pedestrians and vehicular traffic at the junctions of sidewalks and driveways presents a variety of challenges. Some general principles are:

- Consider using right-turn deceleration/storage lanes so that right-turning drivers can safely wait in the auxiliary lane, clear of through traffic, while pedestrians are present in, or near, the driveway (see Appendix D). Queuing length should meet the needs of the design vehicle. In locations where a driveway functions as an intersection, pedestrian safety features should be included. These features may include crosswalks, small corner radii to limit turning speeds to 10 mph, and pedestrian signal heads (if signalized) as warranted.
- Detectable warnings are not required where minor driveways cross a sidewalk. If the driveway functions as a public street crossing, such as commercial driveways with traffic signals, detectable warnings are required.
- Where separator islands are provided between inbound and outbound travel lanes in a driveway, the island must provide pedestrian refuge with a minimum width of 6-ft. See Section 7.3.6.2.1 for the design of pedestrian refuge islands.
- Driveways must maintain the PAR as a continuous, ADA grade-compliant, and clearly delineated path to encourage drivers to stop and yield to pedestrians. For example, if the sidewalk is composed of concrete and it is crossing an asphalt driveway, the concrete should be continuous across the driveway. The PAR within the driveway crossing must meet accessibility standards (see Section 7.3.2.2).
- Locate sidewalks far enough from the curb, or edge of pavement (4-ft or greater is preferred), to provide a suitable vertical curve transition between the pavement cross-slope and the driveway apron and to allow the driveway to cross the sidewalk at the same elevation as the approaching sidewalk.
- Where driveways are closely spaced, consider the use of right-in/right-out driveways to eliminate conflicts between left-turning vehicles and pedestrians. In this case it is recommended that provisions be made for the left-turns only at locations where the vehicular-pedestrian conflict can be safely addressed by appropriate design and traffic control.
- TxDOT may work with local jurisdictions to ensure throat length allows for enough space for vehicles to queue out of the path of pedestrian travel (see Figure 7-18). Vehicles should not block the sidewalk when parked in driveway.

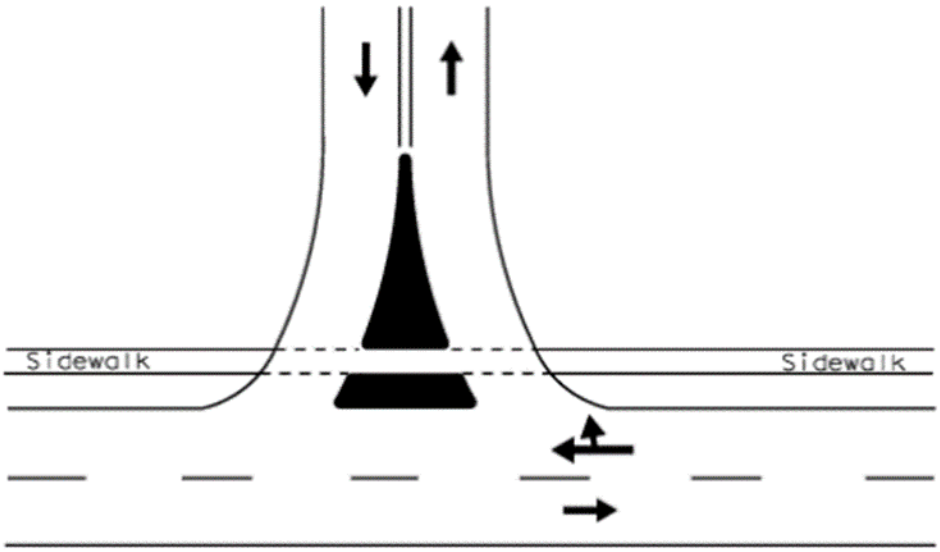


Figure 7-17. Channelizing Island Provide Pedestrian Refuge.

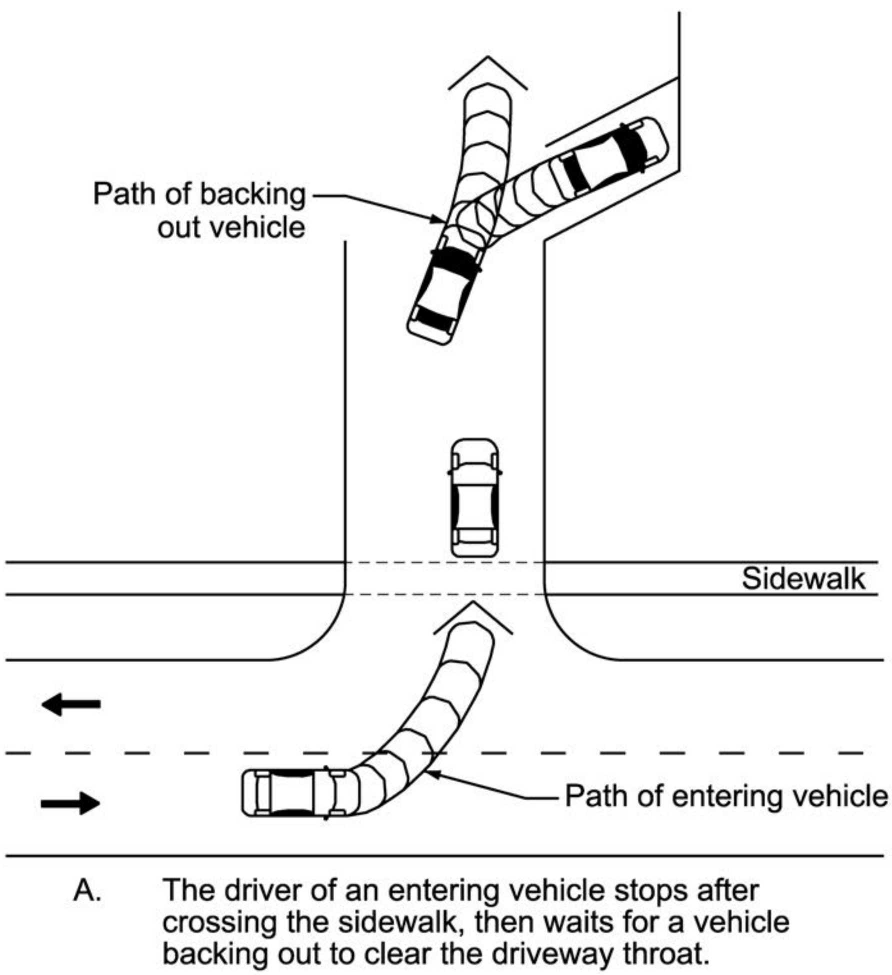


Figure 7-18. Sufficient Throat Length Should Allow Entering Vehicle to Clear the Through Traffic Lane.

7.3.5.2 Designing for Cross Slope Compliance

Sidewalks crossing driveways must comply with the same cross slope requirements (no more than 2 percent) as the rest of the sidewalk corridor. Several layout options exist for compliance with this requirement and are outlined in the following text and accompanying graphic, Figures 7-19 to 7-23. Refer to TxDOT's Pedestrian Facilities Curb Ramps standard drawings for additional guidance on driveways crossing a sidewalk.

1) and 2) These options are preferred, as they maintain the same sidewalk elevation as the approaching sidewalk and reinforces motorist stopping and yielding behavior. In the case of a set-back sidewalk, a width of 4 ft or greater is preferred for the parkway planting to provide suitable apron slopes. See Figures 7-19 and 7-20.

3) This option is preferred if Options 1 and 2 are not feasible. This driveway design provides an increased setback design for the pedestrian accessible route around the driveway space. This drive-way design is long enough to accommodate a car length so as to not prohibit through movement on the roadway. See Figure 7-21.

4) Sidewalks immediately adjacent to the curb or roadway may be offset to avoid a non-conforming cross slope at driveway aprons by diverting the sidewalk around the apron to provide greater dis-tance between the flowline of the gutter and the sidewalk. See Figure 7-22. This option is preferable to Option 5.

5) Where Option 3 is not possible, the sidewalk may slope down across the driveway apron. The sidewalk grade should not be lowered all the way to street level. A 1-in rise at the gutter flowline should be provided so that drainage in the gutter is maintained. Note that sidewalk slopes must meet the provisions of section 405 rather than section 406 of the ADA Standards (2010). See Figure 7-23.

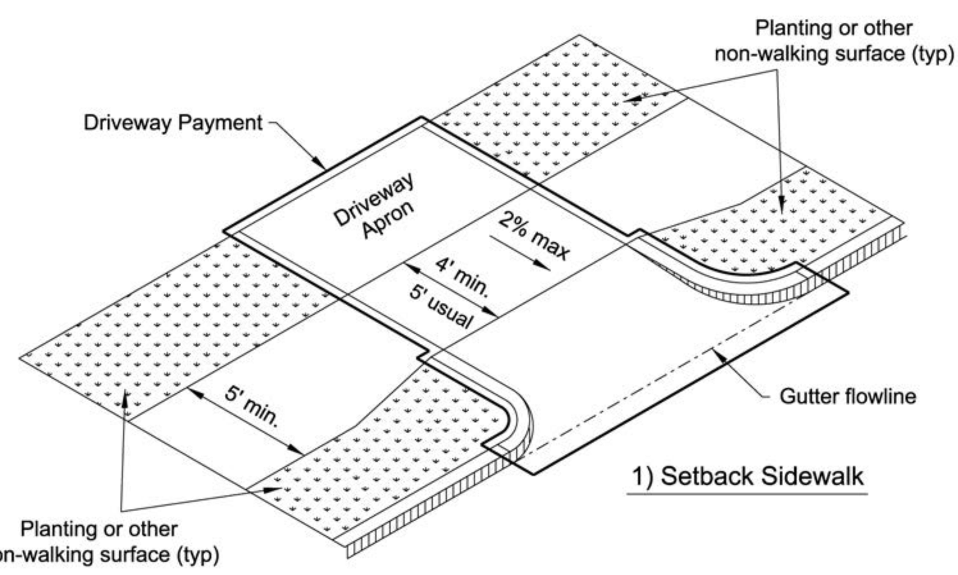


Figure 7-19. Option 1: A Preferred Curb Ramp Design at Driveways.

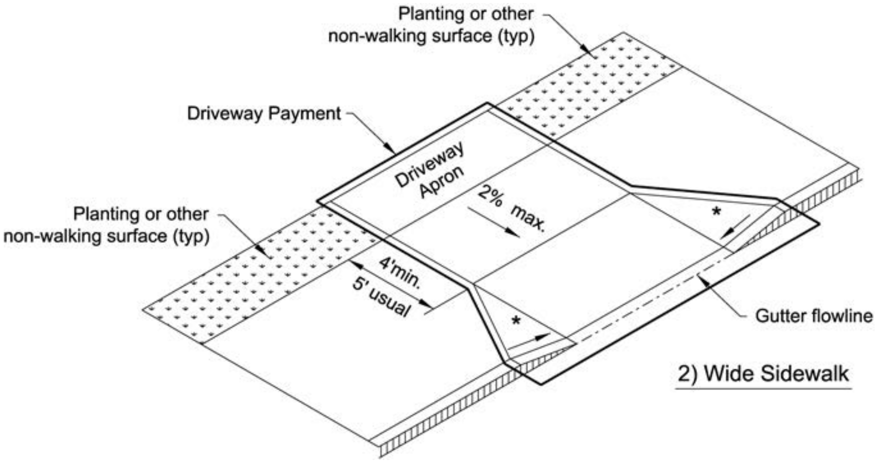


Figure 7-20. Option 2: A Preferred Curb Ramp Design at Driveways.

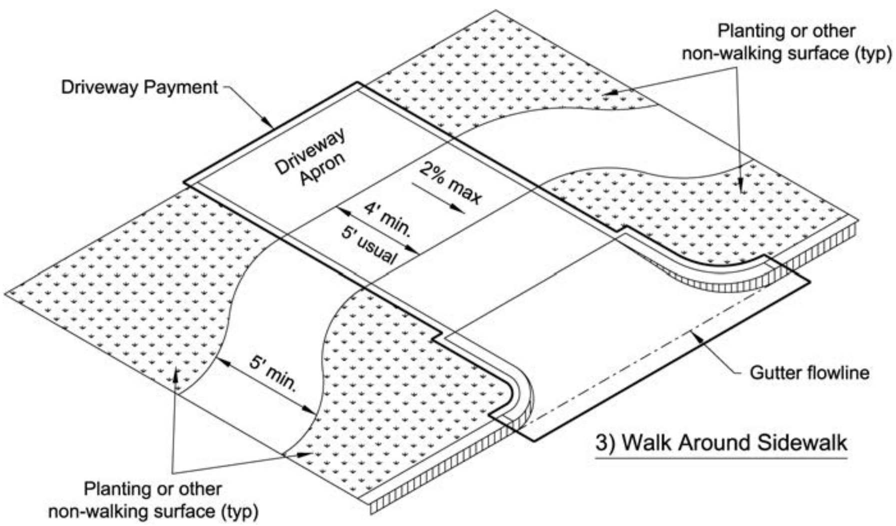


Figure 7-21. Option 3: Walk-Around Design at Driveways.

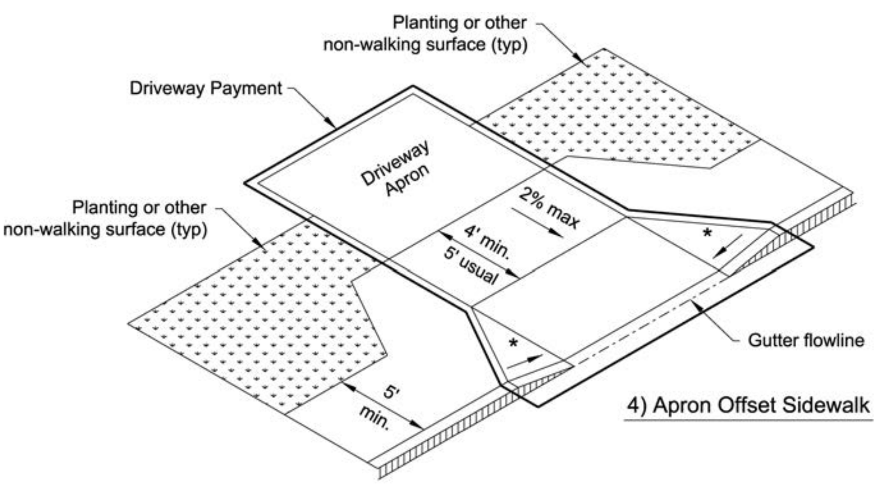


Figure 7-22. Option 4: An Additional Curb Ramp Design at Driveways.
(This design is preferable over Option 5 in Figure 7-23).

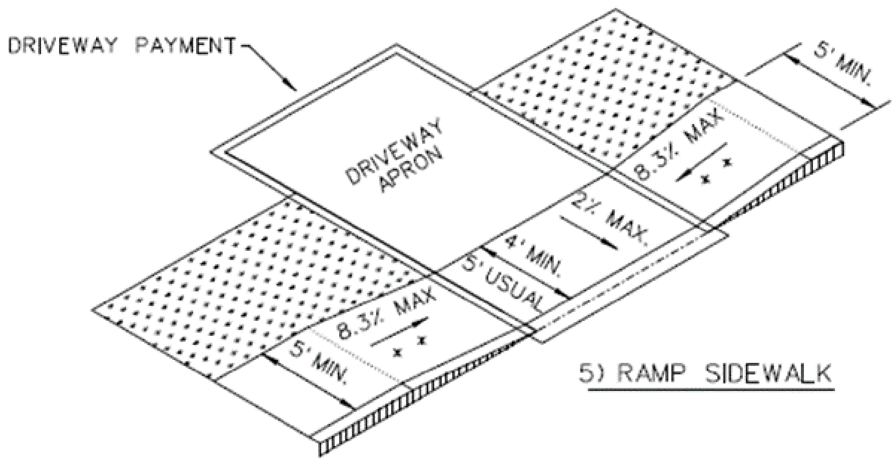


Figure 7-23. Option 5: An Additional Curb Ramp Design at Driveways.

7.3.6 Intersections and Crossings

Designing multimodal intersections requires intersection geometry that increases safety for all users in combination with effective and efficient traffic control measures. Changes in geometry can help to reduce vehicle turning speeds, facilitate safe crossings, increase pedestrian and bicyclist comfort and safety, and create space for dedicated bicycle facilities. One of the key considerations of intersection geometry is the location of pedestrian crossing ramps and crossings relative to vehicle paths. It should be noted that these principles apply to all roadways including frontage roads.

7.3.6.1 Pedestrian Crossings Principles

7.3.6.1.1 Speed of Vehicles

Many pedestrian-vehicle collisions occur at intersections. A key strategy to avoiding these crashes is reducing vehicle speeds, which should be complemented by other strategies, including providing adequate sight distance through sight triangles, providing a variety of signalization strategies at locations with signals, eliminating free-flowing movements (or using truck aprons in slip lanes to slow turning cars while accommodating the larger vehicles), and using infrastructure strategies that increase visibility such as curb extensions and parking restrictions near pedestrian crossings.

Wider curb radii can enable drivers to maintain higher speeds when turning, decreasing reaction time for potential conflicts with crossing pedestrians. Designing to minimum radii accommodates vehicle operating characteristics while providing some control over vehicle speeds.

Traffic volume and vehicle type influence the width and curvature of intersection corner radii. See Chapter 7 Minimum Designs for Truck and Bus Turns.

7.3.6.1.2 Visibility

It is critical that pedestrians and drivers have clear sight lines to one another at crossings, both at intersections and mid-block crossings. This adequate sight distance allows vehicles to stop for crossing pedestrians; higher speeds require longer sight distances. Design elements such as fences, plantings, buildings, and walls may obscure sight lines. Parked vehicles close to crossing locations can also obscure sight lines for both parties. When possible, these elements should be restricted or moved to another location to provide proper visibility. Several treatments discussed below can also improve visibility.

7.3.6.1.3 Frequency of Crossing Opportunities

In areas with high pedestrian volumes and denser land uses, pedestrian crossing opportunities should be closely spaced. "Crossing opportunities" are defined here as intersections or crossings that are equipped with appropriate control, signage, lighting, geometry, and markings that allow pedestrians to safely and comfortably cross the street. In the absence of frequent crossing opportunities, pedestrians will choose the most direct path of travel, thus creating opportunities for collisions with vehicles where pedestrians are unexpected, less visible, and more exposed. In general, the frequency of crossing opportunities should be at least as dense as the surrounding street grid. Where large gaps between crossings are identified, midblock crossing may be appropriate.

7.3.6.1.4 Crossing Distance and Alignment

Reducing street width improves pedestrian safety and comfort by shortening crossing times, reducing exposure to vehicle-pedestrian conflicts, and reducing vehicle delay. Pedestrian crossing distances should be minimized to the greatest extent reasonable. Shorter crossing distances may be achieved through any or a combination of the following treatments:

- Providing median refuge islands
- Providing curb extensions
- Realigning of crosswalks at offset or diagonal intersections
- Reducing vehicle and parking lane widths
- Reducing the number of vehicle lanes

Alignment of crossings is an important aspect to the safe crossing of streets for pedestrians. It is natural for pedestrians to cross the road at convenient locations; therefore, providing crosswalks in those locations that align to minimize distance and time crossing the road is important to consider.

7.3.6.1.5 Crossings at Intersections

Intersections should be designed to enable pedestrians to cross all legs. There may be limited circumstances where pedestrian access across a leg of an intersection may be prohibited, however this should rarely be done unless there is a compelling reason to limit this movement. Pedestrians should not be directed to cross multiple legs of an intersection to access a corner that could otherwise be accessed by crossing a single leg. Crossing additional legs creates more exposure to vehicular traffic for pedestrians and forces pedestrians to travel longer distances.

7.3.6.1.6 Pedestrian Delay

Minimizing pedestrian delay when crossing the street increases a pedestrian's convenience and reduces the likelihood that pedestrians will cross the street in an unsafe manner. At signalized intersections, pedestrian delay can be minimized by maintaining short signal cycles. At uncontrolled crossings, designers should provide pedestrians as much frequency and length of gaps in traffic as possible.

7.3.6.2 Intersection Crossing Treatment Decision-Making Framework

There are many factors that can affect roadway crossing opportunities for pedestrians, including motorist approach speeds and volumes, motorist stopping and yielding behavior, roadway configuration, types of vehicles, the volume and assertiveness of crossing pedestrians, the amount of sight distance that can be achieved at the crossing location, and other factors. These factors will also vary based on time of day or day of the week, seasonally, or due to special events.

Pedestrian crash risk increases as traffic volumes and operating speeds increase and motorist stopping and yielding decreases. To mitigate locations with increased crash risk, countermeasures which increase motorists stopping and yielding, increase gaps, or reduce pedestrian exposure should be considered to facilitate the pedestrian crossing.

The FHWA *Guide for Improving Pedestrian Safety at Uncontrolled Crossing Locations* identifies a range of potential countermeasures which have been shown to achieve improved pedestrian safety at uncontrolled crossings. Figure 7-24 includes an extensive matrix and list of countermeasures which have been found to be effective for improving pedestrian safety at uncontrolled crossing relative to specific combinations of roadway geometry and traffic operating conditions. Other resources which should be consulted to review potential countermeasure not included in this table or within this guide which may be effective include:

- NCHRP Report 562, Improving Pedestrian Safety at Unsignalized Crossings
- NCHRP Report 926, Guidance to Improve Pedestrian and Bicyclist Safety at Intersections
- FHWA Proven Safety Countermeasures website: https://safety.fhwa.dot.gov/provencountermeasures/
- FHWA PedBikeSafe website - http://www.pedbikesafe.org/

Roadway Configuration	Posted Speed Limit and AADT								
	Vehicle AADT <9,000			Vehicle AADT 9,000–15,000			Vehicle AADT >15,000		
	≤30 mph	35 mph	≥40 mph	≤30 mph	35 mph	≥40 mph	≤30 mph	35 mph	≥40 mph
2 lanes (1 lane in each direction)	❶ 2 4 5 6	❶ 5 6 7 9	① 5 6 ❼ ❾	❶ 4 5 6	❶ 5 6 7 9	① 5 6 ❼ ❾	❶ 4 5 6 7 9	① 5 6 7 9	① 5 6 ❾
3 lanes with raised median (1 lane in each direction)	❶ 2 3 4 5	❶ ❸ 5 7 9	① ❸ 5 ❼ ❾	① 3 4 5 7 9	① ❸ 5 ❼ ❾	① ❸ 5 ❼ ❾	① ❸ 4 5 7 9	① ❸ 5 ❼ ❾	① ❸ 5 ❾
3 lanes w/o raised median (1 lane in each direction with a two-way left-turn lane)	❶ 2 3 4 5 6 7 9	❶ ❸ 5 6 7 9	① ❸ 5 6 ❾	① 3 4 5 6 7 9	① ❸ 5 6 ❼ ❾	① ❸ 5 6 ❾	① ❸ 4 5 6 7 9	① ❸ 5 6 ❾	① ❸ 5 6 ❾
4+ lanes with raised median (2 or more lanes in each direction)	❶ ❸ 5 7 8 9	❶ ❸ 5 7 8 9	① ❸ 5 8 ❾	① ❸ 5 7 8 9	① ❸ 5 ❼ 8 ❾	① ❸ 5 8 ❾	① ❸ 5 ❼ 8 ❾	① ❸ 5 8 ❾	① ❸ 5 8 ❾
4+ lanes w/o raised median (2 or more lanes in each direction)	❶ ❸ 5 6 7 8 9	① ❸ 5 ❻ 7 8 9	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ 7 8 9	① ❸ 5 ❻ ❼ 8 ❾	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ ❼ 8 ❾	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ 8 ❾

Given the set of conditions in a cell,

- # Signifies that the countermeasure is a candidate treatment at a marked uncontrolled crossing location.
- ● Signifies that the countermeasure should always be considered, but not mandated or required, based upon engineering judgment at a marked uncontrolled crossing location.
- ○ Signifies that crosswalk visibility enhancements should always occur in conjunction with other identified countermeasures.*

The absence of a number signifies that the countermeasure is generally not an appropriate treatment, but exceptions may be considered following engineering judgment.

1 High-visibility crosswalk markings, parking restrictions on crosswalk approach, adequate nighttime lighting levels, and crossing warning signs
2 Raised crosswalk
3 Advance Yield Here To (Stop Here For) Pedestrians sign and yield (stop) line
4 In-Street Pedestrian Crossing sign
5 Curb extension
6 Pedestrian refuge island
7 Rectangular Rapid-Flashing Beacon (RRFB)**
8 Road Diet
9 Pedestrian Hybrid Beacon (PHB)**

Figure 7-24. Pedestrian Safety Countermeasures Based on Roadway Configuration and Posted Speed Limits and Annual Average Daily Traffic.

7.3.6.3 Geometric Treatments for Pedestrians at Intersections

7.3.6.3.1 Medians and Refuge Islands

Raised medians have been proven to provide safety benefits to pedestrians when compared to flush (at street grade) medians and are therefore preferred as a safety measure at pedestrian crossings. Medians are generally used to divide traffic streams, such as in Figure 7-25 where the midblock median separates two-way traffic and enables the pedestrian to cross one direction of traffic at a time. Channelization islands between turning lanes and through lanes can also act as refuge islands. A two-way center turn lane is not a raised median as it cannot provide safe refuge for crossing pedestrians.

Application

To improve pedestrian safety, refuge islands should be considered where street crossing distances are four travel lanes or greater. See Figure 7-24 in Section 7.3.6.2 for further detail on application with respect to traffic volumes and speeds. Medians and refuge islands also reduce the speed of left turning movements at intersections which can also improve pedestrian safety where they are exposed to left-turning traffic.

Though they provide refuge, median islands should not be used to allow for traffic signal phasing that breaks the pedestrian trip into two phases. Pedestrians should be staged in the median at signalized intersections only on boulevard sections with wide medians that would add excessive pedestrian clearance interval. Where pedestrians are staged in the median, additional pedestrian signal heads and pushbuttons must be installed in the median. For locations with rectangular rapid flashing beacon or pedestrian hybrid beacon (RRFB or PHB), refuge islands may be used to create two-stage crossings.

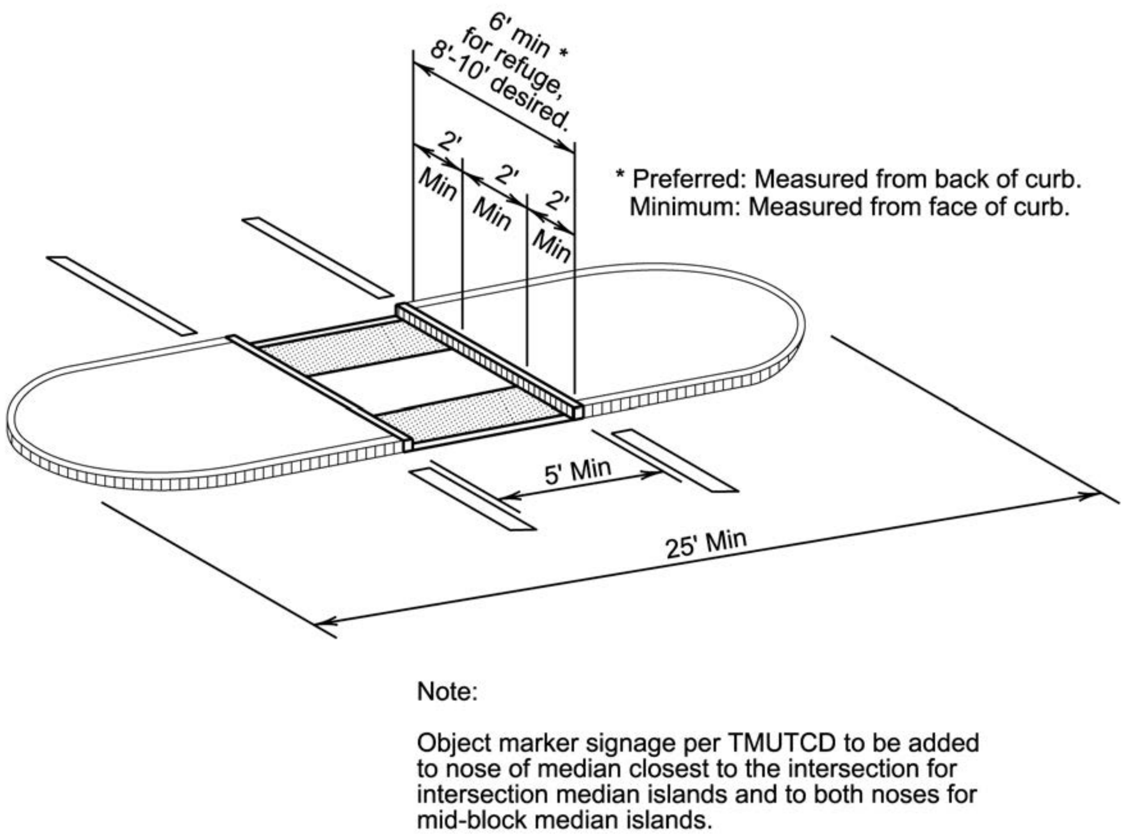


Figure 7-25. A Median Refuge Island.

Design Parameters

Median Width: To function as a refuge (i.e., a place where pedestrians may stop to await a gap in traffic or a signal indication before completing their crossing), a raised median with a pedestrian access route cut through the island at a level flush with the roadway surface must be at least 6-ft wide, exclusive of the width of the curbs. An 8- or 10-ft median island width is desirable to provide additional clearance for wheelchairs and bicycles and to better accommodate groups of pedestrians.

Length: At midblock locations, the minimum length of the median refuge is 25-ft. Advance lane tapers may be required to warn approaching motorists of the physical median following the guidance of the TxDOT MUTCD and PM standard drawings.

Accessibility: If the median is at least 6-ft wide so that it functions as a refuge, then detectable warning surfaces must be installed at the threshold of each roadway surface. The detectable warning surfaces will occupy the same 2-ft needed for clearance from the adjacent travelway, leaving 2-ft open for storage at the center of the island.

PROWAG indicates that if a median island is less than 6-ft wide (measured in the direction of pedestrian travel), detectable warnings should not be installed. An island less than 6-ft wide is not considered adequate for pedestrian refuge and detectable warnings should not be used.

Crosswalk: It is recommended that the cut-through for a median refuge be the full width of the crosswalk in the direction of pedestrian travel. The minimum width of a cut-through for a median refuge is 5-ft. Crossings through a median can be angled so pedestrians can see and be more aware of traffic on the roadway they are about to cross. See Section 7.3.4.5 for curb ramp design at median islands.

At an intersection crossing, a “nose,” or curbed edge, that extends past the crosswalk toward the intersection is recommended to separate people waiting on the crossing island from motorists, and to slow turning motorists. The recommended minimum nose length is 2-ft.

Considerations

Medians and refuge islands can provide additional space for warning signs or beacons at multi-lane pedestrian crossings, increasing their visibility to approaching drivers.

Offset Crossing (“Z-style”) Median Islands

At uncontrolled locations (intersection or midblock), designers may consider a Z-style PAR configuration within the median island. This configuration reorients the pedestrian to face oncoming traffic, creating staggered crosswalks on either side of the island. Islands should be at least 12-ft wide to accommodate an accessible route in both directions and maintain maneuverability and passing distance for pedestrians using mobility devices. When used along a shared use path alignment, the design should also consider maneuverability for bicyclists, including bicycles with trailers or long wheel cargo bikes. This treatment is most appropriate in rural and suburban areas.

7.3.6.3.2 Curb Extensions/Bulb-Outs

Curb extensions are created by extending the sidewalk or curb line into the street at an intersection or mid-block crossing location in order to shorten the crossing distance for pedestrians and improve visibility at crossing locations.

Application

Curb extensions are recommended on streets that have on-street parking and can be used selectively in locations with shoulders where shortened pedestrian crossings are desired. Curb extension installation on both sides of a crossing is preferred, but where curb extension installation on one side is infeasible or inappropriate (i.e., no parking lane or shoulder), this should not preclude installation on the opposite side.

Design Parameters

Width: The width of parking lane minus 1-ft of offset to the travel lane.

Length: The minimum length of a curb extension must be the width of the crosswalk although it can extend farther to match a no parking limit approaching the intersection. The length of a curb extension can also vary depending on the intended use (e.g., stormwater management, transit loading, restrict parking, bike parking) and potential for sight line improvement.

Considerations

In proximity to transit stops, curb extensions can present challenges with bus maneuverability where they must pull to the edge of the roadway in advance of, or after, the curb extension. In these locations, the curb extension should be designed to enable buses to easily navigate to the edge of the roadway or consideration should be given to allowing the bus to stop in the travel lane by incorporating the curb extension into the bus stop, creating a bus bulb.

The radii of the curb extension should be designed so that street-sweeping equipment can reach the entire curb face without leaving gaps where debris can collect. Designers should consult with local jurisdictions regarding the operating characteristics of their sweeping equipment. Landscaping within a curb extension should be limited to low-level plants that will not impact sight distance.

Curb extensions retrofitted onto a street can significantly alter drainage patterns. Drainage must be evaluated as part of the design of both full reconstruction and retrofit curb extensions.

Figure 7-26. A Curb Extension with Landscaping.

7.3.6.3.3 Truck Aprons

A truck apron (Figure 7-27) is a design strategy used to accommodate the turning needs of large vehicles while slowing the turning speeds of smaller vehicles by reducing the perceived actual radius. A truck apron is designed to be mountable by larger vehicles to accommodate their larger effective turning radius needs. The mountable surface encourages the most common vehicles – a passenger (P) or delivery vehicle (SU) to turn outside the apron at a slower speed by design, thereby decreasing their effective turning radius.

Application

Truck aprons are applicable where slower vehicle turning speeds are a priority. They may be used at corners of standard intersections, as part of a channelization island, or as part of the central island of a roundabout. Truck aprons may be used at corners and islands that have either controlled or stop and yield condition vehicle movements. Truck aprons can be installed with corner reconstruction, or in a retrofit condition. When they are installed as a retrofit, or when a gap is left between the mountable curb and the curb face to facilitate surface drainage, they are called truck pillows.

Design Parameters

Color: The apron's color should be easily distinguishable in contrast with the surrounding roadway and sidewalk.

Height: A truck apron/pillow should incorporate a mountable section with a height between 2-in and 3-in to provide the desired traffic calming.

Accessibility: The accessible route across the truck apron should be clear and obvious to pedestrians. The pavement texture and surface slopes at this location should be smooth and meet accessibility standards. While texture is appropriate in other mountable portions of the apron, where the accessible route cuts through, the texture should be discontinued for the portion of the accessible route. Crosswalks should be marked up to the detectable warning surface through the truck apron to clearly distinguish the intended path of travel and have it to be recognized as part of the street. The detectable warning must be located behind the truck apron at the sidewalk.

Considerations

The presence of a higher volume pedestrian or shared use path crossing should be considered as a condition for truck apron installation, especially in an area where drivers may be starting to accelerate, such as a freeway on-ramp.

Figure 7-27. A Truck Apron.

7.3.6.3.4 Raised Crosswalks and Intersections

Sight lines between crossing pedestrians and drivers are essential for safety as are slower motorist approach speeds where they must stop and yield to crossing pedestrians. Placing pedestrians at a higher elevation through the intersection can make them more visible and slow the speeds of approaching motorists. This is especially important at locations where greater numbers of children or people using mobility devices may be present as they are lower to the roadway and can be more difficult to see and may have reduced ability to judge or react to approaching motorists.

Raised Crosswalks

Raised crosswalks continue the elevation of the sidewalk across a crosswalk in order to provide better visibility of and for pedestrians and to slow vehicle speeds in a manner similar to other vertical speed control elements.

Application

Raised crosswalks are possibly applicable in urbanized areas on streets with speeds of 35mph or less. They are also applicable at roundabouts and channelized right-turn lanes between the curb and a triangular island where drivers have a stop and yield condition.

Design Parameters

- *Height:* The crosswalk should be raised approximately 3-in from the elevation of the roadway.
- *Width:* The table of the crosswalk should be at least 10-ft wide, allowing both front and rear wheels of a typical passenger vehicle to be on top of the table at the same time.
- *Approach ramp length:* The recommended length of the ramp up to the raised crosswalk is 8-ft.
- *Markings:* Crosswalk markings are recommended for the top of the raised crosswalk.
- *Accessibility:* The junction between the sidewalk and raised crosswalk should meet all applicable accessibility standards.

Considerations

- Drainage design will differ based upon whether the elevation of the crosswalk extends to the curb or not.
- Chevron-style markings may be used on the half of the ramp facing traffic for increased driver awareness of change in slope.
- Consult with local emergency services providers regarding placement and design.

Raised Intersections

Raised intersections are those where the elevation of the entire center of the intersection is raised. Similar to speed humps and other vertical speed control elements, they reinforce slow speeds and encourage motorists to stop and yield to pedestrians at the crosswalk, while improving visibility for pedestrians and drivers. Crosswalks are typically placed on top of the raised area.

Application

Raised intersections are applicable in urban areas and towns with higher volumes of pedestrians, at minor intersections, on streets with speeds of 35mph or less. They may be used at controlled or uncontrolled locations.

Design Parameters

Accessibility: The junction between the sidewalk and raised crosswalk should meet all applicable accessibility standards.

Drainage: The presence of raised pavement can significantly alter the drainage characteristics of a street. Drainage must be evaluated as part of the design of raised crosswalks and intersections.

Considerations

Raised intersections may have different pavement material to bring attention to the intersection. Ensure that colored pavements or materials still meet ADA requirements for contrast and surface.

7.3.6.3.5 Lower Speed Limits

Motor vehicle speed is known to contribute to crash severity and likelihood of survival for bicyclists and pedestrians. Speed is also a known factor that influences motorist stopping and yielding rates. Lowering the posted speed of roadways can result in the lowering of the operating speeds and improve pedestrian safety and improve motorist compliance with pedestrian safety countermeasures. See the TxDOT *Procedures for Establishing Speed Zones* manual for additional information on posted speed determination.

7.3.6.3.6 Hardened Centerlines

Hardened centerlines are painted centerlines supplemented with vertical barriers at signalized or unsignalized intersections. Hardened centerlines change the path of travel through the intersection, creating tighter left turn radii which improves the visibility of pedestrians in the crosswalk and also reduces turning speed.

Application

Hardened centerlines are applicable in retrofit situations where the intersection has not been otherwise designed to slow turning vehicles and appropriately align them to provide drivers clear sightlines of crossing pedestrians.

Design Parameters

Materials: Materials for hardened centerlines should be sturdy, stable materials, such as plastic bollards and turning wedges. See TxDOT Departmental Materials Specification DMS 4350 for more information. Materials must be listed on the Material Producer List. Install products in accordance with manufacturer's specifications.

Accessibility: Hardened centerlines must not impede pedestrian crossings. Include cut-outs to provide a clear path of travel.

Considerations

The use of hardened centerlines should decrease over time as intersections are built to principles with pedestrian safety in mind or are wholly retrofit with more permanent materials.

7.3.6.4 Uncontrolled Crossing Safety Countermeasures

Uncontrolled crossing locations are those where sidewalks or designated walkways intersect a roadway at a location where no traffic control (i.e., traffic signal or STOP sign) is present. These may be at intersection or midblock locations.

7.3.6.4.1 Marked Crosswalks with Uncontrolled Traffic Movements

As discussed in Section 7.3.1.3.2, motorists must stop and yield the right of way to crossing pedestrians who are lawfully within a marked or unmarked crosswalk at intersections or at marked crosswalks at midblock locations. Marked crosswalks visually communicate to both pedestrians and drivers that pedestrian crossings are expected at that location and remind motorists of their responsibility to stop and yield to pedestrians.

Application

Marked crosswalks indicate optimal or preferred locations for pedestrians to cross, and remind motorists to stop and yield the right of way to crossing pedestrians. They may be installed at any intersection location where it is desired to support pedestrian crossings.

In general, marked crosswalks and other safety treatments should be prioritized at locations where pedestrians desire to cross the street and are vulnerable to conflicts with vehicles such as locations with:

- High pedestrian and vehicular volumes
- Locations where pedestrians are routinely expected typical of urban areas, rural town centers, school zones, parks, university or other similar pedestrian intense institutional land uses, or at bus stops
- Locations with vulnerable populations such as children, senior citizens, people with disabilities, or hospital areas
- Locations where traffic or geometric conditions make it difficult for pedestrians to cross creating safety challenges for pedestrians
- Locations with shared use path or trail crossings

Marked crosswalk research shows that they are an effective treatment to increase motorist awareness of pedestrians and to improve understanding of right-of-way compared to unmarked crosswalks. However, this research also shows that marked crosswalks used alone may increase pedestrian crash risk if not used with other supplemental engineering treatments at uncontrolled crossings where the speed limit exceeds 40 mph and either:

- A. The roadway has four or more lanes of travel without a raised median or pedestrian refuge island and an ADT of 12,000 vehicles per day or greater; or

- B. The roadway has four or more lanes of travel with a raised median or pedestrian refuge island and an ADT of 15,000 vehicles per day or greater.

At these locations, it will be necessary to install other measures to reduce traffic speeds, shorten crossing distances, enhance driver awareness of the crossing, and/or provide active warning of pedestrian presence to supplement the marked crosswalk following the guidance in Section 7.3.6.2.

A marked crosswalk is required to legally establish the midblock crossing per Texas code§541.302.

Design Parameters

Marked crosswalks must follow TxDOT PM standards and further guidance found in TMUTCD Section 3B.18.

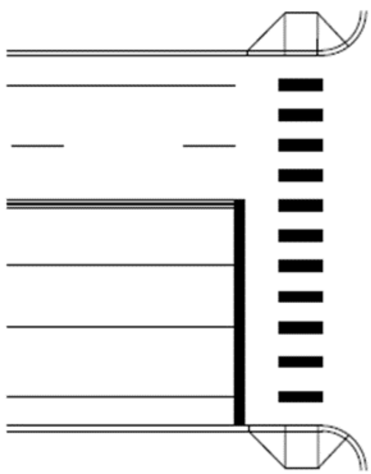


Figure 7-28. A Marked Crosswalk

Stop Line Location: Stop lines should be used to indicate where vehicles are required to stop in advance of a marked crosswalk at signalized and stop controlled intersections. Locating stop bars at least 6-ft from the crosswalk can increase pedestrian comfort and may decrease the likelihood of encroachment on the sidewalk by vehicles. The minimum separation is 4-ft.

Considerations

Supportive Traffic Control Signs: Regulatory and warning traffic signs (see Section 7.3.6.4.3) may be used at and in advance of a crosswalk where pedestrians may be unexpected, and it is desired to improve visibility of the crossing.

Standard markings will incur wheel wear and need to be maintained over time. Longitudinal cross-walk markings should not be in the wheel path of vehicles. Center the crosswalk lines on travel lanes, lane lines, and shoulder lines (if present) to reduce, wear, and thus maintenance. To provide better visibility on concrete roadways, the white crosswalk markings may be outlined with black contrast markings.

7.3.6.4.2 Stop Bars and Yield Lines

Stop bars, yield lines, or advance yield markings, improve safety at uncontrolled crossings on multilane roadways by improving visibility and decreasing the possibility of a multiple threat collision. These lines reinforce a driver's responsibility to stop and yield to pedestrians in a crosswalk at an uncontrolled location, or stop and yield condition movement within a signalized intersection, e.g., a channelized right turn lane. Stop bars, or yield lines are those placed in advance of a marked crosswalk. See TMUTCD Section 3B.16 and Figure 3B-17.

Application

Refer to Figure 7-24 in Section 7.3.6.2 for application. Because they are effective at mitigating multiple threat crashes, they should be considered at all marked, unsignalized crossings of multilane roadways. They may be used on single lane approaches to marked crosswalks where engineering judgment determines a need.

Design Parameters

See Section 3B.16 of the TMUTCD and the current PM standard for further design guidance.

Considerations

Parking should be restricted on both sides of the street between the stop bars, and yield line markings and the marked crosswalk to improve visibility. See Section 3B.16 of the TMUTCD for further guidance.

Stop bars may be staggered across lanes if appropriate.

"PED XING," "SCHOOL XING," or "TRAIL XING" pavement markings may be placed in advance of the stop bar pavement markings to identify the purpose of the stop bars is a crossing along with a corresponding warning sign.

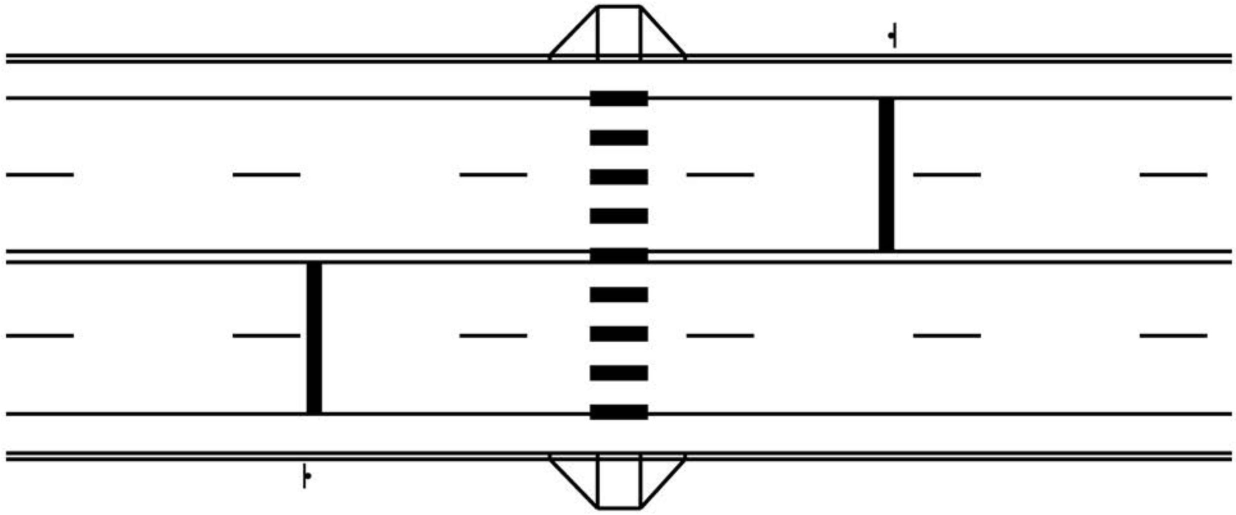


Figure 7-29. Stop Bars at a Marked Crosswalk

7.3.6.4.3 Supportive Signs

The functions of signs are to provide regulations, warnings, and guidance information for road users. They can be supplemented with markings. Any information intended for pedestrians must be accessible to all pedestrians. The following discussion focuses on the most common signs needed to support engineering treatments in this chapter. The TMUTCD describes the use and application of other signs not discussed which may be beneficial to accommodating pedestrians.

Wayfinding is discussed in section 7.3.9 below.

7.3.6.4.4 Pedestrian Crossing Warning Sign Assemblies

Application

Pedestrian traffic warning signs should be used at and in advance of a marked or unmarked crosswalk to notify motorists of the crossing and improve visibility of the crossing as a baseline treatment to marked crosswalks. Refer to Figure 7-24 in Section 7.3.6.2 for application. The message should reflect the type of crossing (pedestrian (W11-2), school (S1-1), trail (W11-15 or W11-15a)). They should be used at all marked, uncontrolled crosswalks. See TMUTCD Section 2C.50 for guidance and standards for use. School zone traffic control is further covered in Chapter 7 of the TMUTCD.

Figure 7-30. Pedestrian Crossing Warning Signs (left to right): Pedestrian Crossing, School Crossing, Pedestrian and Bike Trail Crossing, and Trail Crossing.

Considerations

Signs may be mounted back to back on the same sign post to allow double posting of signs on the left and right side of the road to improve visibility to approaching motorists. Signs should be such that visibility is not obscured by other signs, utility poles, buildings, or vegetation.

An advanced placement of these signs should be considered in locations where sight distance is obstructed, or the crossing may be unexpected following the guidance of TMUTCD Section 2C.05.

Beacons or flashers can be used to supplement warning signs and can be mounted on the sign post with the warning sign or over the crossing. They can flash continuously or be actuated by a pedestrian waiting to cross; however, these signs have been shown to be more effective if actuated. Actuation can occur when a pedestrian uses a pushbutton or through passive detection such as infrared. Wherever pedestrian actuated systems are installed, they should be accessible.

7.3.6.4.5 Pedestrian Crossing Regulatory Sign Assemblies

Application

The “Stop Here for Pedestrians” series signs (R1-5b, R1-5c, R1-6a, and R1-9a) may be used to notify approaching motorists of their responsibility to stop and yield to crossing pedestrians at uncontrolled crossings. Refer to Figure 7-24 in Section 7.3.6.2 for application. See Section 2B.11, 2B.12, and Section 3B.16 of the TMUTCD for further standards and guidance.

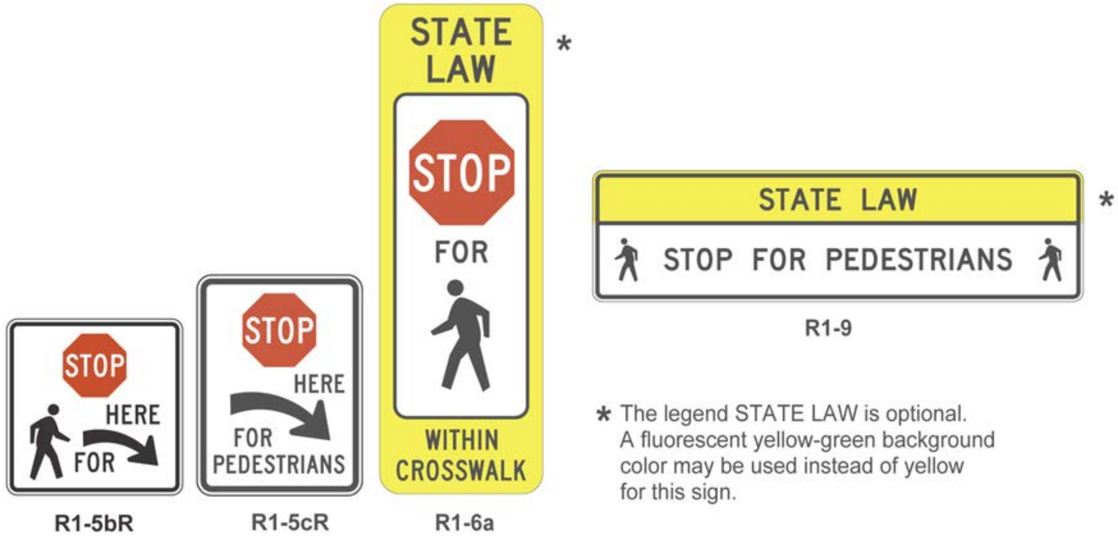


Figure 7-31. Unsignalized Pedestrian Crossing Signs.

Figure 7-32. Pedestrian Stop Sign at an Uncontrolled Intersection.

Considerations

In-street Stop for pedestrian sign (R1-6a) have been found to be effective at increasing motorist stopping rates on roadways operating at 30 mph or less with ADT of 25,000 or less (see Figure 7-32).

7.3.6.4.6 Rectangular Rapid Flashing Beacon

A rectangular rapid flashing beacon (RRFB) is a device that supplements a warning sign that consists of amber LEDs that use an irregular flash pattern to draw roadway users' attention to a crossing when a pedestrian is present. They may supplement any of the warning signs discussed in Section 7.3.6.4.3. RRFBs are generally actuated with a pedestrian push button but may also be activated through passive detection.

As of September 11, 2018, RRFBs have interim approval for use across Texas though they do not appear in the current (2011) edition of the TMUTCD. Contact Traffic Safety Division for specific guidance on the use of RRFBs. (As of the publishing of this manual, the link for interim approval of rectangular rapid flashing beacons can be found here: https://ftp.dot.state.tx.us/pub/txdot-info/trf/pdf/revised-guidelines.pdf.)

Application

RRFBs can be used at and in advance of an uncontrolled marked or unmarked crosswalk to notify motorists of the crossing and improve visibility of the crossing as a baseline treatment at locations where static signs or flashing beacons are not sufficient to induce motorist stopping and yielding. Refer to Figure 7-24 in Section 7.3.6.2 for application.

Considerations

On multi-lane roadways, consider the visibility of roadway edge signs to drivers in the middle lanes. Where a median island is present, placement of additional RRFB assemblies there may increase visibility and thus stopping and yielding compliance. RRFBs may also be placed in advance of the crossing on higher speed roadways or at locations with sight distance restrictions providing motorists sufficient time to react to the device to slow and stop and yield to crossing pedestrians. The advance sign(s) can be designed to wirelessly communicate when the crossing is actuated.

The R1-5b signs and advanced stop bar markings may also be used with the RRFB. Care should be taken such that R1-5b signs do not restrict motorists' view of the flashing beacon at the crosswalk.

Solar-powered assemblies can eliminate the need for connecting to a power source.

Figure 7-33. Rectangular Rapid Flashing Beacon.

7.3.6.5 Signalized Crossing Safety Countermeasures

7.3.6.5.1 Warrants

Warrants specified in Chapter 4C of the TMUTCD govern the installation of traffic signals at intersections, including provisions to accommodate pedestrians. Engineering studies to support the evaluation of traffic signal warrants must include a review of pedestrian characteristics and volume and whether the location is a school crossing.

7.3.6.5.2 Standard Signals

Application

Traffic signals provide benefits to pedestrians by stopping automobile traffic and allowing a designated phase for pedestrians to cross a roadway. See Section 7.3.6.2, and the Traffic Signals Manual for further guidance on the use of traffic signals.

Consideration

The needs of all pedestrians should be taken into account when designing traffic signals at intersections where they can be expected to cross. Pedestrian safety, comfort, and convenience at intersections is fundamentally impacted by several major design decisions including cycle length, crossing time, and phase selection. Signalized intersections should be designed for pedestrians to cross in one phase. Wait times for pedestrians at traffic signals should be minimized. Substantial delays encourage pedestrians to violate the traffic signals and cross against Don’t Walk signals. Phasing based on other factors (volume, anticipated turning volumes, etc.) should be reviewed for its adequacy in accommodating pedestrian travel across each intersection leg. Long cycle lengths which dramatically increase pedestrian delay during periods of lower traffic volumes can reduce pedestrian compliance rates inducing attempts to cross during gaps in traffic. This is a particular concern during evening hours when pedestrian fatalities are highest.

7.3.6.5.3 Pedestrian Hybrid Beacon

A pedestrian hybrid beacon (PHB) is a pedestrian-activated warning device located on the roadside, or on mast arms over midblock or intersection pedestrian crossings. The beacon head consists of two red lenses above a single yellow lens. The beacon head is dark until the pedestrian wanting to cross the roadway presses the button and activates the beacons.

This device provides an additional tool for improving the safety of crosswalks when traffic signals do not meet warrants. PHBs should be used in conjunction with signs and pavement markings to warn and control traffic at locations where pedestrians enter or cross a street or highway.

Districts must receive Traffic Safety Division (TRF) approval for installation of a PHB for each location.

(As of the publishing of this manual, the link for interim approval of rectangular rapid flashing beacons can be found here: https://ftp.dot.state.tx.us/pub/txdot-info/trf/pdf/revised-guidelines.pdf.)

Application

Refer to Figure 7-24 and Chapter 5 Section 7 of the Traffic Signals Manual for application. PHBs are generally more applicable on wider, higher volume roadways where a stop and yield condition does not provide reliable, safe, frequent opportunities for pedestrians to cross.

All of the following conditions must be met for a PHB to be considered:

- An engineering study must be performed and meet the guidelines detailed in Chapter 4F of the TMUTCD;
- An established crosswalk with adequate visibility, markings and signs;
- A posted speed limit of 40 mph or less (does not include school speed zones);
- 20 pedestrians or more crossing in one hour;
- Location deemed as a high-risk area (e.g. schools, shopping centers, etc.); and
- Crosswalk is more than 300 ft. from an existing, traffic controlled pedestrian crossing.

See Chapter 4F of the TMUTCD for specific design standards and guidelines.

Considerations

Designers may make the case for a PHB in a location that today does not have 20 pedestrians or more crossing in one hour. (See considerations below) If a location may draw that level of pedestrian traffic once the PHB is implemented, or if there are anticipated nearby land use changes that will attract more significant volumes of pedestrian traffic.

Figure 7-34. A Pedestrian Hybrid Beacon.

7.3.6.5.4 Use of Pedestrian Signal Faces

Application

Pedestrian signal faces aid in crossing by communicating timing and permission specific to pedestrian travel who may be crossing at a traffic signal or a Pedestrian Hybrid Beacon.

Pedestrian signal heads should be provided at all signalized crossings with sidewalks and curb ramps on the approaches, as well as at all signalized crossings where pedestrian activity is expected, regardless of the presence of sidewalks. See Chapter 4E of the TMUTCD for guidance and standards, including for accessible pedestrian signals (APS).

Considerations

Marked crosswalks should be installed at all crossings with pedestrian signals. See Section 7.3.6.4.1for details.

Pedestrian signal faces may be timed and phased differently from the general traffic signal face, thus better accommodating the differing travel characteristics and speeds of pedestrians. See Section 4E.03 of the TMUTCD for further specifications regarding the application of pedestrian signal faces.

7.3.6.5.5 Actuation

Pedestrian actuation may impact signal or beacon operation in two ways: 1) actuating pedestrian beacons or signal heads that direct pedestrians' crossing behavior, and/or 2) it may also change overall operations of a signal to extend a pedestrian phase only when pedestrians are present, thus allowing them to cross the full width of an intersection leg in one phase.

Pedestrian Pushbuttons

Push buttons allow pedestrians to actuate a signal or beacon by pressing a button. Some emerging technologies allow pedestrians to activate the signal by waving a hand in the vicinity of the button as well. See TMUTCD Sections 4E.08 through 4E.13 for guidance and standards regarding use of pushbuttons.

Application

Actuation is optional at signals but required for pedestrian hybrid beacons and RRFBs. All buttons should be accessible.

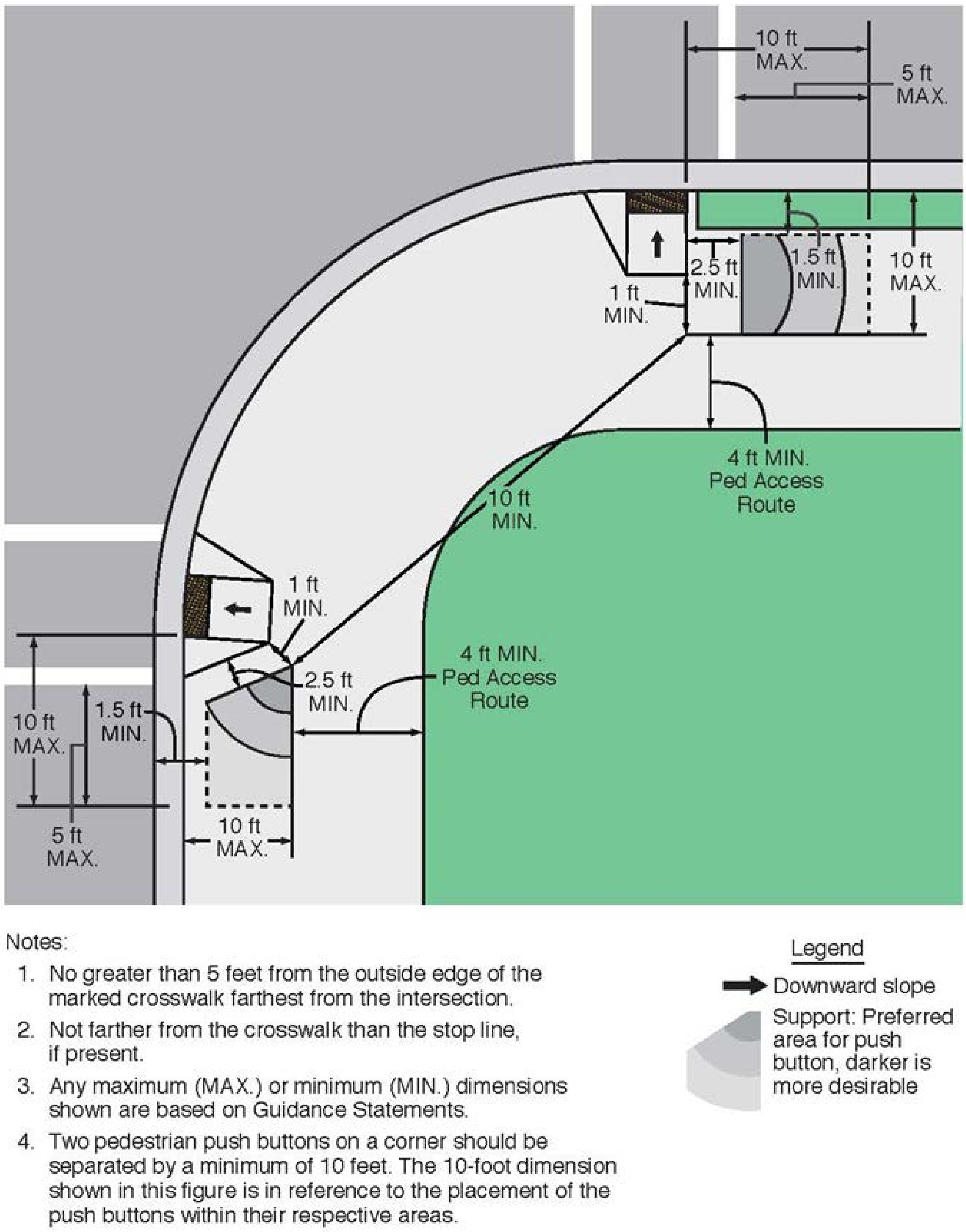


Figure 7-35. Preferred Pushbutton Locations. (Source: Draft MUTCD, Federal Highway Administration). Pedestrian Actuation Pushbuttons. Source: Draft MUTCD 2021 Update, FHWA.

Considerations

If the button is mounted on its own pole, the pole should not obstruct the pedestrian access route. Where there are physical constraints that make it impractical to place the pedestrian pushbutton adjacent to a level all-weather surface; the surface should be as level as feasible.

Though it is not desirable to design for an intersection where pedestrians are most likely to cross in two stages (curb to median, median to curb), where signal design necessitates this operation, a pushbutton must be provided in the median refuge for pedestrians to activate the pedestrian signal heads for the second crossing.

The use of audible or visual confirmation of detection communicates to pedestrians that the WALKING PERSON indication is forthcoming can improve signal compliance. This can be provided with an accessible signal.

Signal supports may be excluded from clear zone requirements. Refer to Chapter 2 Section 7, Clear Zone and Table 2-12 for further considerations when placing signal supports within the clear zone.

Passive Detection

Passive detection systems consist of one of several types of technologies (e.g., infrared, microwave, etc.) to detect pedestrians' presence near the point of departure at a crosswalk.

Application

Passive detection may be used to activate pedestrian phases and signal heads and/or to supplement other sources of actuation by adjusting pedestrian clearance time when a pedestrian is detected.

7.3.6.5.6 Timing Considerations

Signal timing may be designed or amended to create a safer, more comfortable pedestrian environment through several strategies. Signalized intersections are common locations for pedestrian-vehicle collisions, most often with turning vehicles; collisions can occur between parties traveling the same direction (right-turn conflict) or in opposing directions (left-turn conflict). Collisions may also occur between pedestrians and vehicles traveling in perpendicular directions when pedestrians do not have adequate crossing time and remain in the roadway once the signal changes, or when either party fails to obey traffic controls. The strategies below can aid in avoiding these conflicts.

Protected Turn Phasing

Signal phasing that includes protected left-, right- or both turn phases can benefit pedestrian safety by removing potential conflict through separating pedestrian and vehicle travel in time. Pedestrians crossing an intersection leg proceed with the through movement in the same direction while turning traffic is held.

Application

Protected phasing should be considered where pedestrian volumes are anticipated to be high, where there are high percentages of child or older pedestrians, and at locations with multiple conflict point turn lanes across a crosswalk. Protected phasing should also be considered where intersection geometry allows vehicles to turn in excess of 20 mph while executing turning movements. The pedestrian crossing interval must meet the requirements of Section 4E.06 of the TMUTCD.

Considerations

Application of protected turn phasing to an existing signal should be accompanied by the installation of an accessible pedestrian signal so pedestrians with disabilities will follow the same behavior as other pedestrians. Intersections with right and left turn lanes and pedestrian crossing islands create the opportunity to create exclusive pedestrian phases by restricting motorists turning movements to shared right and left turn overlaps. The exclusive pedestrian crossing can be provided concurrent with through moving motorists.

During periods of lower traffic volume protected phases could be adjusted to permissive with flashing yellow arrows for right and/or left turns across pedestrian crossings. This can provide flexibility to reduce conflicts during the pedestrian walk phase at key times during the day or during special events limiting times of protected phases to periods with the greatest need.

Leading Pedestrian Intervals

A leading pedestrian interval (LPI) activates the white WALK indication for the crosswalk while a red indication continues to be displayed for vehicle traffic (through and/or turning) in the same direction. This allows pedestrians to proceed into the crosswalk in advance of vehicle movements, thus making them more visible to turning drivers who may cross their path of travel. They are a proven FHWA Safety Countermeasure. See Section 4E.06 of the TMUTCD for further guidance.

Application

An LPI should be considered for locations with high pedestrian volumes and high volumes of turning vehicles or locations with a history of crashes between pedestrians and turning motorists.

Design Parameter

If a leading pedestrian interval is used, they should be timed to allow pedestrians to cross at least one lane of traffic or, in the case of a large corner radius, to travel far enough for pedestrians to establish their position ahead of the turning traffic before the turning traffic is released. A typical LPI interval range is between 3 and 7 seconds with a minimum of 3 seconds. There is no maximum interval, however long intervals may be better served by completely protected crossing phases.

Considerations

Application of an LPI to an existing signal should be accompanied by the installation of an accessible pedestrian signal so pedestrians with disabilities will follow the same behavior as other pedestrians.

If a leading pedestrian interval is used, consideration should be given to prohibiting turns across the crosswalk during the leading pedestrian interval. See Section 7.3.6.5.6, “Motorist RTOR Restrictions” for additional information.

7.3.6.5.7 Supportive Signs

Crosswalk Restrictions

Intersections should be designed to enable pedestrians to cross all legs. There may be limited circumstances where pedestrian access across a leg of an intersection may be prohibited, however this should rarely be done unless there is a compelling reason to limit this movement. Pedestrians should not be directed to cross multiple legs of an intersection to access a corner that could otherwise be accessed by crossing a single leg. Crossing additional legs creates more exposure to vehicular traffic for pedestrians and forces pedestrians to travel longer distances. If pedestrian crossings at unmarked crosswalks must be restricted, sign types R9-2, R9-3, or other appropriate regulatory signs must be installed to face pedestrian approaches of restricted crosswalks. Also note that prohibited crossing locations must be blocked by landscaping or other physical means to prevent unintended use by pedestrians who are blind or have low vision.

R9-3

Figure 7-36. Restricted Pedestrian Crossing Sign.

Motorist RTOR Restrictions

Right Turn on Red (RTOR) introduces pedestrian safety concerns because drivers scanning for gaps in traffic on their left may not look for pedestrians on their right. Allowing drivers to turn right or left during a pedestrian WALK signal is a frequent cause of crashes between pedestrians and drivers and can impede a pedestrian's ability to complete their crossing by delaying their entry into the roadway. At locations which allow RTOR, motorists are likely to encroach into the crosswalk while watching oncoming vehicles, further eroding pedestrian safety and comfort. These conflicts can be reduced by restricting RTOR movements. The FHWA Pedestrian Safety Guide and Countermeasure Selection System suggests that "prohibiting RTOR should be considered where exclusive pedestrian phases or high pedestrian volumes are present."

Right turn-on red can be restricted when activated by a pedestrian push button by using NO RIGHT ON RED blank-out signs to limit the restriction to times when pedestrians are present.

Turning Traffic Stop and Yield

Left and right turning motorists at signals can account for a high percentage of conflicts and crashes with pedestrians at locations with permissive phasing. Where conflicts between turning motorists and crossing pedestrians are likely, a TURNING TRAFFIC STOP FOR PEDESTRIANS (R10-15a) sign should be considered. This sign may be used for both left and right turning traffic.

7.3.6.6 Mid-block Pedestrian Crossing Safety Countermeasures

Midblock pedestrian crossings may be appropriate in a variety of contexts based on pedestrian desire lines, transit stop locations, land use context, and intersection spacing. Motorists are more likely to expect pedestrians at intersection locations and often are driving at higher speeds in mid-block locations. Because of this, the use and design of midblock crossings should be deliberate to address pedestrian safety and improve motorist compliance.

Given the differences between intersection and midblock crossings, there are several key considerations designers must keep in mind:

- The crossing location should be convenient for pedestrians;
- Motorists should be alerted of the crossing as they approach it;
- Pedestrians must be able to assess opportunities to cross; and
- All users must be made aware of their responsibilities and obligations at the crossing and designers must provide opportunities to meet those responsibilities and obligations.

Designers should consider pedestrian volumes, motorist volumes, types of vehicles, traffic speeds, roadway characteristics (e.g. number of travel lanes), and adjacent land use context when determining if a midblock crosswalk should be provided. Additionally, pedestrians have a strong desire to stay on their path of travel and do not want to go unnecessarily out of their way to utilize a crossing, so crossing locations should be placed at or near the pedestrians' desired path of travel.

7.3.6.6.1 Location Considerations

To promote and achieve high compliance, midblock crossings should be located where intersection spacing is excessive and there is a natural desire line for the pedestrians' path of travel. These conditions may include some of the following:

- Signalized intersection spacing exceeds 600-ft;
- A shared use path intersects a street midblock;
- Transit stops are located midblock;
- Existing pedestrian demand demonstrates a need (e.g. parking lot and office building on opposite sides of the roadway); and
- New or existing development generates heavy pedestrian traffic (e.g. shopping center, school, etc.).

7.3.6.6.2 Crossing Treatments

The guidance in Sections 7.3.6.3 through 7.3.6.5 for pedestrian crossing treatments and countermeasures is similarly applicable to midblock crossings. See latest standard PM(4) and the TMUTCD for guidance on additional signing and markings for midblock crossings.

7.3.6.7 Pedestrian Facilities at Alternative Intersections

Pedestrian facilities should be included at alternative intersections using the same design principles that apply for standard intersections and midblock crossings. For alternative intersection treatments, adequate sight distance is particularly important as vehicles and pedestrians are often approaching one another at non-right angles. Refer to Appendix E for further guidance on these intersections.

7.3.6.8 Pedestrian Treatments at Railroad Crossings

Texas has a network of railroad systems, which are operated and controlled by federal, local, and state laws and regulations. Where the rail networks cross pedestrian facilities, it is important to follow specified guidance that makes the crossing safe and predictable for the pedestrian and rail operator. All pedestrian facilities should be designed to minimize pedestrian crossing time, and devices should be designed to avoid trapping pedestrians between sets of tracks.

- Traffic Control. The TMUTCD has guidance on signage, gates, flashing beacons and other rail crossing warning devices to communicate with pedestrians at railroad crossings. Traffic studies may be needed to determine the level and type of traffic control that should be used. See TMUTCD for more guidance.
- Passive and Active Devices. May be used to supplement highway-related active control devices to improve non-motorist safety at highway-rail crossings. These devices should be considered at crossings with high pedestrian traffic volumes, high train speeds or frequency, extremely wide crossings, complex highway-rail crossing geometry with complex ROW assignment, school zones, inadequate sight distance; and/or multiple tracks. Passive or Active Devices should always be used if no highway-related active control devices are present.
  - Passive devices include fencing, swing gates, pedestrian barriers, detectable warning surfaces, pavement markings, texturing, refuge areas, and fixed message signs.
  - Active devices include flashers, audible active control devices, automated pedestrian gates, pedestrian signals, variable message signs, and blank-out signs. Railroad operating practices may not support installation of pedestrian only devices.
- See Federal Rail Administration’s Highway-Rail Crossing Handbook and Engineering Design for Pedestrian Safety at Highway-Rail Grade Crossings report for further detail on these treatments.
- Visibility. Clear sightlines should be established so that pedestrians can clearly see oncoming trains and assess whether there is adequate time to cross.

- Approach Angle. Perpendicular crossings are preferred in order to provide adequate sight lines and decrease the risk of mobility device wheels getting stuck in flangeway gaps. Where the roadway crossing is skewed, consider widening or providing a bend out in the sidewalk at the rail crossing that allows for a crossing angle closer to perpendicular, preferably between 60 and 90 degrees.
- Flangeway Gaps. PROWAG recommends flangeway gaps should not exceed 3-in for freight tracks or 2.5-in for non-freight tracks
- Detectable Warning Surfaces. Detectable warning surfaces (DWS) such as truncated domes, should be placed on either side of the crossing and outside of the train's "dynamic envelope," the space the train takes up on either side of the track. The distance between the rail and end of the dynamic envelope is 6 feet unless otherwise stated by the operating railroad. PROWAG and AASHTO recommend the following design guidance for DWS placement:
  - The nearest edge of the DWS to the rail should be a minimum of 6 feet and a maximum of 15 feet.
  - The detectable warning should be, at minimum, 2 feet wide in the direction of pedestrian travel and shall extend the full width of the crossing, including any flared sides or ramps that may be present.
- Quiet Zones: Note that specific standards apply to pedestrian crossing treatments and signage for designated quiet zones. See Federal Railroad Administration materials related to quiet zones here: https://railroads.dot.gov/highway-rail-crossing-and-trespasser-programs/train-horn-rulequiet-zones/train-horn-rule-and-quiet

More details on design options and treatments can be found in the *Federal Rail Administration's Highway-Rail Crossing Handbook* Chapter 2 and report "*Engineering Design for Pedestrian Safety at Highway-Rail Grade Crossings*."

7.3.7 Overcrossings and Underpasses

7.3.7.1 Sidewalks for Bridges and Underpasses

7.3.7.1.1 General

While most bridges and underpasses primarily serve motor vehicle traffic, they may also represent the only nearby option for pedestrians to cross a significant barrier. To serve the needs of pedestrians, grade-separated crossings should consider accessibility, user comfort, and personal safety, including preserving good sight lines.

Pedestrian facilities should be provided on both sides of vehicular bridges and underpasses in all area types. See below for design parameters specific to bridges and to underpasses. Bridges and underpasses with PAR should consider lighting for personal comfort and safety for users. Further guidance about lighting is available in Section 7.3.10.

7.3.7.1.2 Bridges

A PAR should be provided as part of on-system bridges on both sides of vehicular bridges in urbanized areas, if only as an emergency exit path. In urban and suburban areas, this walking area should consist of a sidewalk and/or shared use path

Wherever possible, sidewalk widths across bridges should be the same or greater than the clear width of the existing connecting sidewalks. A greater width provides increased pedestrian comfort by allowing people to walk further from moving traffic.

7.3.7.1.3 Underpasses

Pedestrian facilities such as sidewalks must be provided at vehicular underpasses. Where underpasses are constructed without pedestrian facilities, design should anticipate and provide space for pedestrian facilities to be constructed in the future.

Wherever possible, sidewalk widths through underpasses should be the same as the clear width of the existing connecting sidewalks. See Ch. 2 for vertical clearance requirements for pedestrian crossover structures.

Underpasses that are below grade should provide clear sight distances to and through the underpass. Drainage must be carefully considered to maintain accessible use of underpasses. Overhead clearance for pedestrians is much less than clearance for vehicles, so it is possible to lessen the grade for sidewalks at below grade underpasses. This will make the sidewalk more useable and can increase safety as well.

Lighting should be considered for pedestrian facilities within underpasses. This lighting can be placed along the sidewalk buffer zone or attached to the overpass structure.

7.3.7.2 Railings and Handrails for Pedestrian Facilities

Railings and handrails can improve safety and comfort for pedestrians on bridges/overpasses and in underpasses, and on pedestrian/bike bridges or in pedestrian/bike underpasses. In cases where railings or handrails are provided on both sides of a sidewalk or shared use path, clear width must be maintained between them as measured from the face of the railing. The minimum clear width between railings is 4-ft and passing spaces of 5-ft are required every 200-ft. However, designers should consider consistent wider clear spaces for pedestrian facilities on all bridges and underpasses as they need to accommodate the potential for two users passing one another; a 5-ft passing space between two adjacent vertical elements is not adequate for two users in mobility devices to navigate past one another.

Additionally, pedestrians' natural behavior is to shy away from vertical elements, such as railings or walls, adjacent to a pedestrian route, and the area of the sidewalk they tend to avoid is referred to as shy distance. A shy distance of 2-ft is recommended for pedestrian facilities on bridges and underpasses where continuous railings or handrails are used, with a minimum of 1-ft shy distance in constrained locations where lane widths and other widths are also reduced to minimum dimensions. This may result in a wider sidewalk than the facility approaching the bridge or underpass.

7.3.7.2.1 Railings

Railings may be used on the outside edge of a pedestrian facility on a bridge, between the pedestrian facility and adjacent travelway on a bridge or in an underpass, or on the edges of a pedestrian/ bike bridge. Railing height and placement requirements differ based on the expected design user, the type of facility, and the adjacent slope. The warranting conditions where railings should be considered adjacent to steep slopes are discussed below under section 7.3.12. See Chapter 3 Section 2 of the *Bridge Railing Manual* and Chapter 3 Section 1 of the *Bridge Design Manual* for further guidance on the use of railings on bridges.

Figure 7-37. A Railing as a Barrier Against a Steep Drop-Off for a Sidewalk Facility at an Overpass.

7.3.7.2.2 Handrails

To maintain minimum ADA requirements on sloped approaches to a grade-separated facility (bridge or overpass), handrails must be provided at a continuous height between 34- and 38-in. TxDOT's Pedestrian Handrail standard is for handrail height of 36-in above the walk surface. A second set of handrails at a maximum height of 28-in may be considered if children are expected to be regular users of the facility. If two levels of handrails are provided, the minimum vertical clearance between the two should be 9-in to reduce the likelihood of entrapment. See TAS Section 505 for additional design criteria related to handrails.

7.3.7.3 Pedestrian/Bike Bridges and Underpasses

Where a designer is not able to provide appropriate treatments for the at-grade pedestrian crossing of a roadway, or where the nature of the barrier being crossed does not allow an at-grade crossing (e.g., freeway, water body, depressed rail line, etc.), a grade-separated crossing should be considered. It is not practical to develop warrants governing the construction of pedestrian grade separation facilities. Each situation must be considered on its own merits.

The need for a grade-separated crossing must include an engineering analysis of all alternatives, including safety improvements to nearby at-grade crossings. When the construction of a pedestrian grade separation is considered, an investigation should be made including studies of pedestrian crossing volumes, type of highway to be crossed, proximity of adjacent crossing facilities, the predominant type and age of persons who will use the facility, and the cost of constructing the pedestrian grade separation.

The effectiveness of grade-separated crossings depends on their perceived ease of accessibility by pedestrians. An overpass or underpass will not be used simply because it improves safety. Pedestrians tend to weigh the perceived safety of using the facility against the extra effort and time needed. The degree of use of overpasses and underpasses by pedestrians depends on walking distances and the convenience of the facility.

The primary location for pedestrian overpasses/underpasses is often an urbanized area but may be appropriate in other contexts. Such a pedestrian crossing may be considered when the following conditions exist:

- Pedestrian demand to cross a freeway or expressway is moderate to high;
- A large number of children need to regularly cross a high-speed, high-volume roadway (particularly near schools);
- Conflicts that would be encountered by pedestrians are considered unacceptable (e.g., on wide streets with high pedestrian crossing volumes combined with high-speed traffic); or
- One or more of the conditions stated above exists in conjunction with a well-defined pedestrian origin and destination (e.g., a residential neighborhood across a busy street from a school; a parking structure affiliated with a university or other campus; a high-volume, multi-use trail; a major entertainment destination; or an apartment complex near a shopping mall).
- Grades on either side of the proposed bridge or underpass are conducive to building the structure

Figure 7-38. Underpasses For Pedestrians And Bicyclists Should Provide Clear Sightlines At The Approach And Through The Underpass.

A pedestrian grade separation should only be constructed when the need for the safe movement of pedestrians cannot be provided in a more efficient manner. Additional guidance concerning pedestrian grade separations can be found in an AASHTO publication *Guide Specifications for Design of Pedestrian Bridges*.

7.3.7.3.1 Pedestrian/Bike Bridges

Design Parameters

Grade: Bridges must comply with ADA guidelines: not exceeding a 5 percent running slope or 2 percent cross slope. Where space constraints result in necessary steeper grades, bridges must be designed to a maximum slope of 8.3 percent, and landings are required for every 30-in of ramp rise. Landings must be a minimum of 5-ft in length and can have a maximum 2 percent slope in any direction. Assuming a full 30-in rise at 8.3 percent and a 5-ft landing at 2 percent slope results in an "effective grade" of 7.4 percent. It is recommended that this be used for short, constrained segments only if needed rather than a basis of design for long, sustained segments.

Width: The minimum inside clear width of pedestrian bridges, exclusive of railings and handrails, is 8-ft. If the adjacent sidewalk widths are greater than 8-ft, then the minimum inside clear width must match the adjacent sidewalk widths. A minimum width of 14-ft is recommended if the pedestrian bridge will be shared with bicycle traffic. Where width of a bridge is sufficient to permit vehicle passage, designs should include some type of deterrent with the preferred choice being a forgiving bollard since more robust bollards may present safety issues for pedestrians and bicyclists. Greater deterrents are recommended where the structure would not support the weight of a vehicle.

Vertical Clearance: See Ch. 2 for vertical clearance requirements for pedestrian crossover structures.

Lighting: Adequate lighting should be provided for pedestrian and bike bridges that allows users to see one another in nighttime conditions to avoid collisions and improve the perception of personal security.

7.3.7.3.2 Pedestrian/Bike Underpasses

Design Parameters

Grade: Underpasses must comply with ADA guidelines: not exceeding a 5 percent running slope or 2 percent cross slope. Where space constraints result in necessary steeper grades, underpasses must still comply with ADA requirements. Underpasses' grades and lengths should allow for clear sightlines at the approach and through the underpass.

Width: Width recommendations are 14- to16-ft, but underpasses longer in length and expecting high use in urban areas should be wider. Where width of an underpass is sufficient to permit vehicle passage, designs should include some type of deterrent with the preferred choice being a forgiving bollard since more robust bollards may present safety issues for pedestrians and bicyclists.

Vertical Clearance: A minimum vertical clearance of 10-ft is required. Designers should consider additional height when it would provide clear sight lines through the underpass,

Lighting: Adequate lighting should be provided for pedestrian and bike underpasses that allows users to see one another in nighttime conditions to avoid collisions and improve the perception of personal security.

7.3.8 Work Zone and Temporary Traffic Control Pedestrian Accommodations

Texas Administrative Code Title 43 §22.12 states that traffic control plans shall address pedestrian traffic during individual phases of construction and operation. Part 6 of the TMUTCD provides guidance on the planning and design of temporary traffic control (TTC) which states, "The needs and control of all road users (motorists, bicyclists, and pedestrians within the highway, or on private roads open to public travel … including persons with disabilities in accordance with the Americans with Disabilities Act of 1990 (ADA), Title II, Paragraph 35.130) through a TTC zone shall be an essential part of highway construction, utility work, maintenance operations, and the management of traffic incidents." Refer to the TMUTCD and the WZ (BTS-2) standard for more information on specific traffic control measures for pedestrians in work zones.

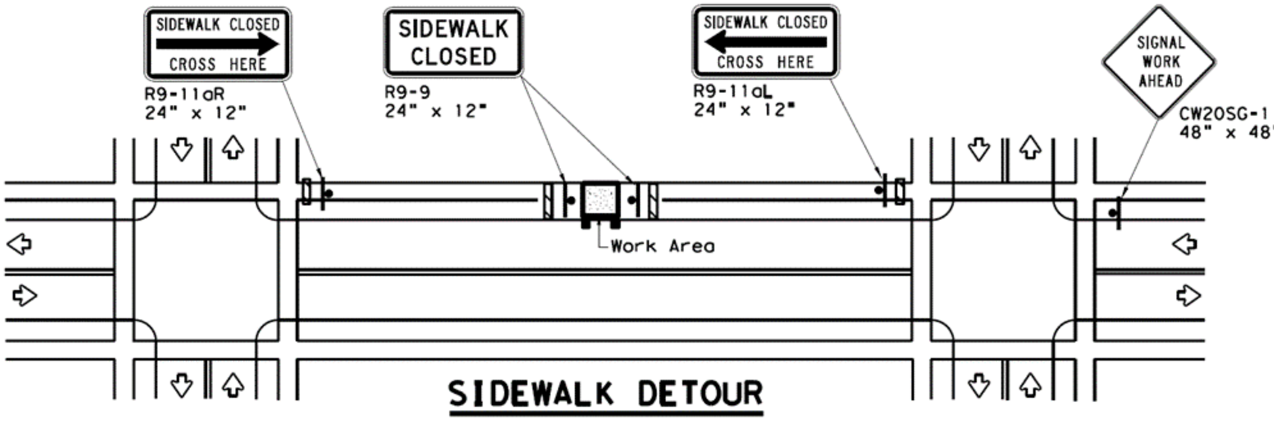


Figure 7-39. Sidewalk Detour Details.

Construction activities that affect pedestrian facilities or connectivity to those facilities must provide and maintain an accessible detour. In some cases, this route may continue to be along the roadway in question, e.g., creating a temporary walking path by placing appropriate barricades, reflectorized drums, and signage to reallocate roadway space during closure of a sidewalk. Walkways must be clearly identified, accessible, and free from hazards. Barricades separating the pedestrian route from motor vehicle traffic must be of sufficient strength to withstand intrusion by an impacting vehicle. On higher-speed roadways, continuous, temporary pre-cast concrete barriers are recommended.

It is preferred to maintain the pedestrian route on the same side of the street when a sidewalk is closed. Completely closing a sidewalk for construction and rerouting pedestrians to the other side of the street is another, less desirable option. In the event pedestrians are rerouted to the other side of the street, the street crossing should be located at an intersection, existing crosswalk, or other suitable and safe location. Access to pedestrian pushbuttons should be maintained during sidewalk closures.

Other cases may completely close a roadway that provides pedestrian access to destinations in areas of high pedestrian activity, e.g., the closure of a bridge or underpass across a freeway.

See TMUTCD Section 2D.50 for additional guidance on the placement of pedestrian oriented wayfinding signage.

7.3.9 Lighting

Illuminating sidewalks and crossings makes it easier for people to see when walking at night and also increases their visibility to drivers. There are many considerations when selecting, designing, and implementing lighting to enhance the pedestrian environment. The presence of pedestrian-generating land uses and existing or anticipated pedestrian crossing activity should be considered when determining the appropriate level of illumination. In general, areas with greater anticipated pedestrian volumes, such as commercial districts, near parks and schools, and along transit routes, are most in need of lighting.

All locations where pedestrian crossing is expected and planned for, including signalized intersections, uncontrolled intersections, and mid-block locations, should have adequate lighting at the crosswalk. At controlled and uncontrolled intersections, luminaires should be located no more than10-ft from the crosswalk and positioned to light the side of the pedestrian facing the approaching vehicle.

TxDOT follows FHWA standards for illumination levels and uniformity for lighting for roadways, walkways, bicycle facilities, crosswalks, and pedestrian underpasses. Illumination levels vary depending on luminaire type, pole height, roadway type, and level of pedestrian activity or conflict. Preferred pedestrian lighting layouts will vary based on the functional classification and cross-section being illuminated. Designers should refer to Chapter 6 Section 2 of the *Highway Illumination Manual* for illuminance values for pedestrian facilities and intersections.

Lighting should be designed to avoid unnecessary light spillover, energy usage, and environmental impacts. Quality and color temperature of light can impact the character and visitor perception of a street or neighborhood. However, special caution should be used when selecting a color temperature for lighting in and adjacent to residential areas. Higher color temperatures produce a bluer light that can disrupt human sleep cycles.

Although various light sources are approved, Light Emitting Diode (LED) lighting is encouraged, as it is more energy efficient. Lighting is typically located in the buffer zone. Lighting should be oriented towards both the roadway and the sidewalk, ensuring that both are illuminated. All street lighting should include no uplight fixtures (BUG rating with U=0) which direct light downward and reduce the potential for glare and light trespass on adjacent properties.

Pedestrian lighting can be used alone or in combination with roadway scale lighting in high activity areas to accommodate pedestrian activity at night. Pedestrian lighting can be located on the same pole as roadway lighting to reduce the number of poles within the Street buffer zone lighting. Lighting on rural roadways is generally not required, but should be considered for intersections, roundabouts, and areas where pedestrians and bicyclists may be present.

Lighting should be placed at least 24-in behind the face of the curb. Poles should be placed in accordance with guidance regarding obstructions in Section 7.3.2.2.1.

Design of lighting systems should be coordinated with local utilities.

7.3.10 On-Street Parking

Where on-street parking is provided, it should be provided in accordance with PROWAG's guidance on accessible parking spaces including the minimum number of accessible parking spaces required. All accessible parking spaces shall be marked with the International Symbol of Accessibility that contrasts with the pavement surface. Parking spaces and access aisles shall be designed so that vehicles, when parked, cannot obstruct the required clear width of adjacent pedestrian access routes.

Additional guidance for on-street parking is best taken from the PROWAG Sections R214 (the number of accessible spaces), and R309 (dimension requirements).

Additional guidance for facilities such as safety rest areas, buildings, and parking facilities is found in TAS Section 208 Parking Spaces.

7.3.10.1 Parallel Accessible Parking

Parallel accessible spaces adjacent to a sidewalk greater than 14-ft wide require an access aisle. Access aisle must be minimum 5-ft wide, run the length of the parking space, and be connected to the pedestrian access route by a curb ramp between the roadway and sidewalk. Additional pavement markings are not required for the access aisle at parallel parking spaces, but access aisles shall not encroach on the vehicular travel lane. Parallel accessible spaces adjacent to sidewalks less than or equal to 14-ft do not require an access aisle if the space is located at the end of the block. Sidewalks 8-ft wide or wider adjacent to parallel parking spaces should be kept clear of obstructions to allow for wheelchair deployment. Signs shall be located at the head or the foot of the parking space. The sidewalk adjacent to accessible parallel parking spaces should be free of signs, street furniture, and other obstructions to permit deployment of a van side-lift or ramp or the vehicle occupant to transfer to a wheelchair or scooter.

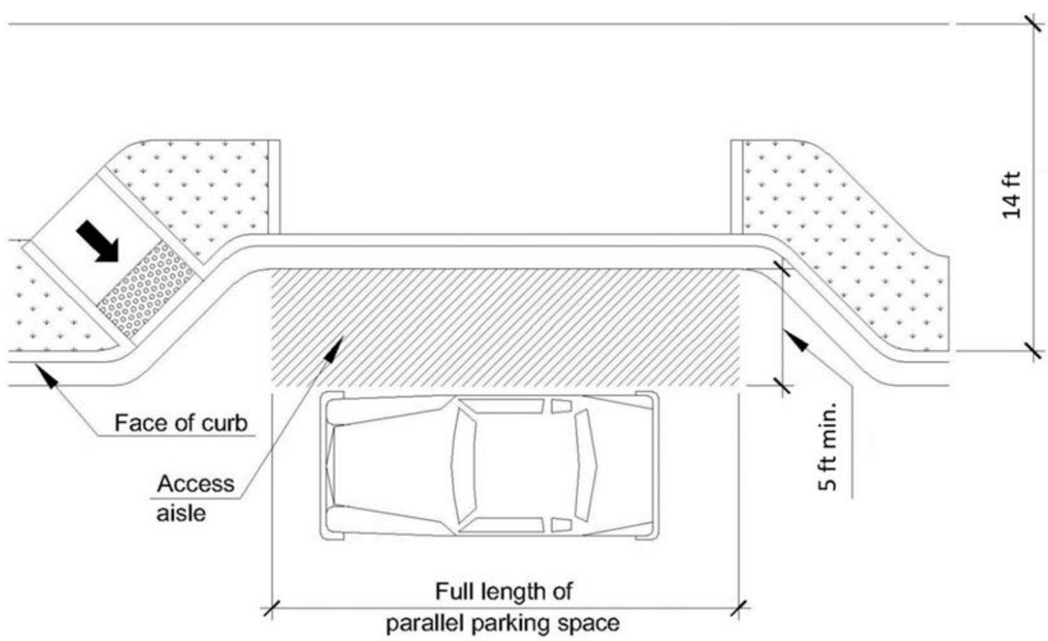


Figure 7-40. Accessible Parallel Parking

7.3.10.2 Perpendicular Accessible Parking

Perpendicular/Angled accessible spaces require a van accessible aisle with a minimum width of 8-ft. Two adjacent perpendicular or angled accessible spaces may share a common access aisle. The upcoming Traffic Standard PM(AP) provides specific guidance for pavement markings and signing for accessible parking.

7.3.10.3 Access Aisles

Access aisles shall be at street level, extend the full length of the parking space they serve, follow ADA Surface requirements (PROWAG R309.2), and be connected to the adjacent pedestrian access route with a curb ramp. Curb ramps shall not be located within the access aisle. Detectable warnings are not required on curb ramps that connect to access aisles unless they also serve pedestrian street crossings.

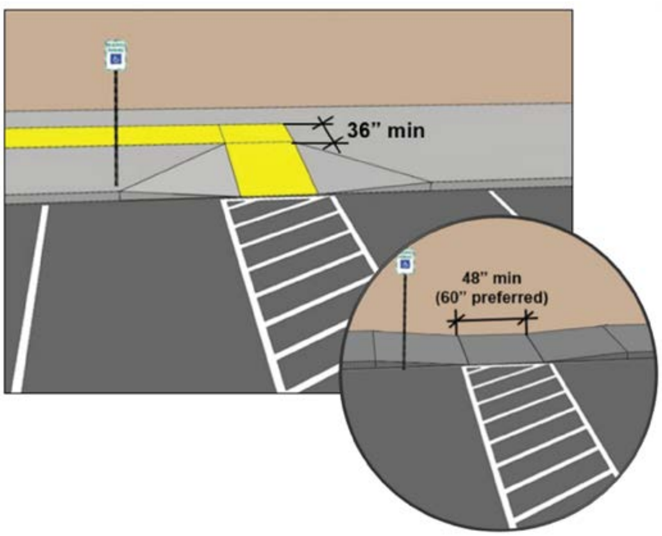


Figure 7-41. Perpendicular Parking Access Aisles, Showing A Curb Ramp With Flares (Top) And A Parallel Curb Ramp (Bottom).

7.3.11 Transit Access

Transit riders are also pedestrians for the portion of their trip to and from the transit stop. As with other pedestrian facilities, transit stops must be accessible to all members of the traveling public. Transit stops must comply with Section R308 of PROWAG. Transit-stop design, including amenities and location, should be determined in consultation with the transit operators.

Boarding and alighting areas for transit stops require a minimum 5-ft width (parallel to the street) and minimum 8-ft length (perpendicular to the street) that is hardscaped and free of obstructions. Consider providing wider areas to allow for pedestrian passage around boarding, alighting, and waiting transit passengers. The slope of the boarding area perpendicular to the roadway must be no more than 2 percent, but cross slope may follow the general grade of the roadway. Stops must be located such that the front door of the bus aligns with this landing area.

Boarding areas must be connected to streets, sidewalks, or pedestrian circulation paths by an accessible route.

Bus shelters, if provided, must also connect to the boarding area by an accessible route. Transit shelters must be connected by a PAR to boarding areas. Transit shelters must provide a minimum clear space of at least 2.5-ft x 4-ft entirely within the shelter; this clear space must comply with other provisions of Section R404 of PROWAG. Where seating is provided within transit shelters, the clear space must be located either at one end of a seat or must not overlap the area within 1.5-ft from the front edge of the seat. Protruding objects within transit shelters must comply with Section R402 of PROWAG.

Transit stops should be well lit and highly visible to improve the sense of safety and comfort at all times of day. Consider seating at or near transit stops. Seating need not be a unique and dedicated element, but may include leaning rails, planters, ledges, or other street elements. Street furniture associated with transit stops is typically provided by the transit agency and/or local jurisdiction.

7.3.12 Railings Adjacent to Steep Slopes

In some instances, there may be steep slopes adjacent to the walkway if the roadway is elevated relative to the adjacent roadside, next to an open drainage ditch, or on an approach to a highway bridge. In these situations, engineering judgment should be used to assess the need for a physical barrier such as a railing or fence to separate pedestrians from the adjacent drop-off. Depending upon the height of the embankment, the slope of the adjacent roadside, and the conditions at the bottom of the slope, barriers or rails are recommended if any of the following situations occur within 2-ft of the edge of the sidewalk:

- Slopes between 1V:3H and 1V:2H, with a drop of 6-ft or greater (Figure 7-42)
- Slopes of between 1V:3H and 1V:2H, adjacent to a parallel body of water of 2-ft or greater depth, or other substantial obstacle (Figure 7-43)
- Slopes of between 1V:2H and 1V:1H, with a drop of 4-ft or greater (Figure 7-44),or
- Slopes of 1V:1H or steeper, with a drop of 1-ft or greater (Figure 7-45)

Railings used for drop-off protection should be 42-in minimum height. Rail construction should be consistent with AASHTO’s LRFD Bridge Design Specifications per the *Bridge Railing Manual*.

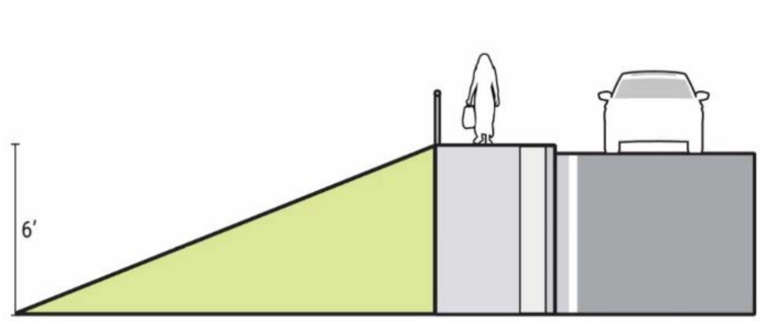


Figure 7-42. Slopes Between 1V: 3H and 1V:2H With a Drop of 6-ft or Greater.

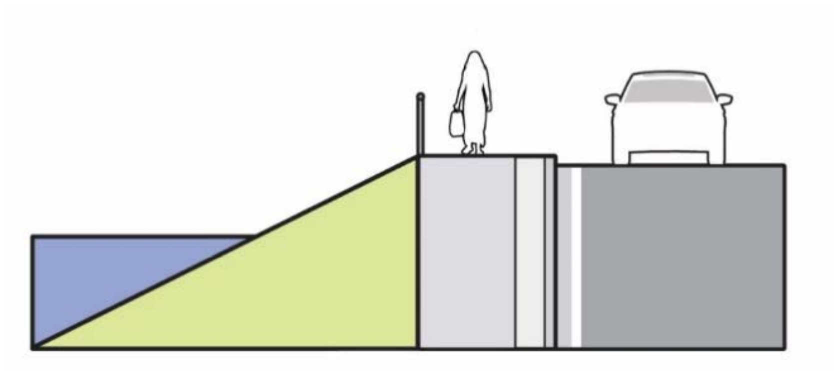


Figure 7-43. Slopes of Between 1V:3H and 1V:2H, Adjacent to a Parallel Body of Water or Other Substantial Obstacle.

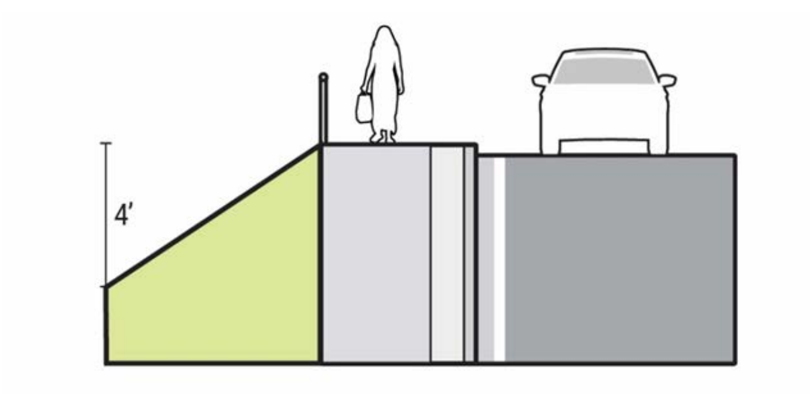


Figure 7-44. Slopes of Between 1V:2H and 1V:1H, with a Drop of 4-ft or Greater.

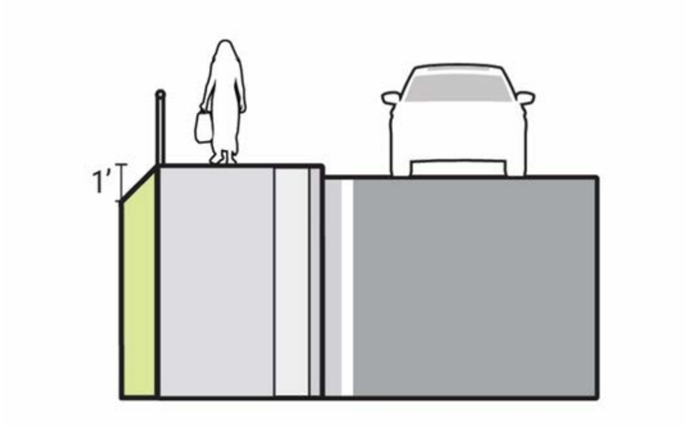


Figure 7-45. Slopes of 1V:1H or Steeper, with a Drop of 1-ft or Greater.

7.3.13 Additional Considerations

7.3.13.1 Personal Security

People may not choose to walk for their trip if they do not feel personally secure along the entirety of the route. While not all aspects of personal security can be impacted by roadway design, there are some elements that can be made more secure through design. For example, ensuring the pedestrian access route is well lit will have a positive impact on personal security (see section above). Simply providing an attractive pedestrian experience thereby encouraging more pedestrian activity improves personal security by bringing more "eyes on the street."

Where a diversity of active land uses is not present or regular volumes of pedestrians are not expected, open sight lines can promote a sense of security. It is important to reduce or eliminate barriers or blind spots where pedestrians cannot be seen by others, or by motorists traveling along the road. Vegetation in the public right-of-way should be regularly maintained to promote open sight lines., Landscaping can be used in the buffer zones for pedestrian comfort and safety and can also be used to help direct them to established street crossings. Landscape plantings should preserve sightlines between the sidewalk and roadway through appropriate choice of the height and type of plantings.

7.3.13.2 Aesthetics

Attractively designed pedestrian facilities can lead to higher usage by making pedestrians feel integrated and welcomed in the transportation system. Pedestrians' slower travel speeds mean that they appreciate and expect a greater diversity of roadside elements. Landscaping, whether with vegetation or varying construction materials (e.g., colored concrete, pavers) in and adjacent to the sidewalk corridor, can enhance the pedestrian experience. Maintenance of vegetation and materials should be considered in the project development process since unmaintained landscaping can eventually become detrimental to the pedestrian experience. See the *Landscape and Aesthetics Design Manual* for further guidance.

7.3.13.3 Exposure to the Elements

Adverse weather conditions present a barrier to pedestrian travel. Protection from heat, sun, and rain should be considered in the project design process as this protection enables more consistent use of pedestrian facilities. Shelters and trees can provide protection from the elements. Additionally, properly designed drainage and regular maintenance will keep pedestrian facilities usable without pooling water in the event of precipitation.

Plantings should be selected according to guidance in TxDOT's *Landscape and Aesthetics Design Manual*.

7.3.14 Micromobility Vehicles

Micromobility vehicles include small, fully or partially human-powered vehicles such as bicycles, e-bikes, e-scooters, and others. They are mostly provided in urban areas in Texas but may also be found in some suburban areas. These vehicles may either be privately owned or part of fleets that are publicly available for short-term rentals. Fleet vehicles are most often parked in the right-of-way for pick-up and drop-off either at stations (docked systems) or within a service area (dockless).

In urban and suburban areas serviced by dockless micromobility vehicles, it is recommended to identify adequate width outside of the PAR for parking of the micromobility vehicles. This device parking area may be located in the buffer zone of the sidewalk corridor or within the street adjacent to the curb. The area may flex over time to be used for standard bike parking or other uses if future needs change. Coordination with the local government and micromobility companies should be done to best determine the sizes and locations of the device parking areas. The boundaries of the parking areas should be clearly identified for the use of micromobility vehicles with surface markings or signage. Many micromobility companies are also now including photo verification of appropriate parking within their smart phone applications which encourages users to properly park vehicles outside the PAR.

In areas serviced by docked micromobility vehicles, proposed locations for new docking stations must be coordinated with the local government according to its requirements for micromobility vehicles. The local government must be notified of any projects affecting existing docking stations. Concerns of the local government should be addressed in the project to the extent practical.

An accessible PAR must be provided around and adjacent to micromobility docking stations and parking areas, per accessibility guidelines.

Section 4 — Parking

Overview

This section discusses the features and design criteria for parking and includes the following subsections:

- Fringe parking lots and
- Parking along highways and arterial streets.

Fringe Parking Lots

Fringe parking lots are congestion mitigation and energy conservation measures utilized by TxDOT. Depending on the function which they are intended to serve, they may be one of the following types of facilities:

- Park and pool lots;
- Park and ride lots; and
- Combination park and pool/park and ride lots.

Park and Pool Lots

Park and Pool lots are usually located on the fringe of an urban area along an arterial roadway at a convenient point where a group of two or more drivers from a surrounding area can gather, leave their individual vehicle and proceed to a common destination in one of the group member's vehicles. The carpool may consist of two or more persons per vehicle. The lot may provide space for a small to large number of vehicles and serve many carpools involving several destinations.

Park and pool lots are located within the highway right-of-way except where they may be in combination with a park and ride lot as discussed below. They are eligible for Federal-aid participation. The lots should be simply designed to accommodate the passenger vehicle with regard to parking stall widths, drive through isles and turning movements.

Park and Ride Lots

Park and Ride lots are generally constructed along express bus routes and are designed to intercept automobiles from low-density suburban developments along transitway corridors. The quality of transit service must be attractive. The time required to reach the destination point by bus must be comparable to or better than driving one's own vehicle.

The facility should be located with regard to the following criteria:

- Along a corridor which encounters 20,000 vehicles per day, per lane;

- In advance of the point where intense traffic congestion routinely occurs;
- 4 to 5 miles from the activity center (usually the Central Business District) served by the transitway and at least 4 to 5 miles from another park and ride facility;
- Downstream from, but in the immediate area of, sufficient demand for travel to the activity center being served; and
- On the right-hand side of the inbound roadway.

Other desirable features include the following:

- Good accessibility to the adjoining street system;
- No parking fees;
- Space for future expansion; and
- Fencing.

Typical park and ride layouts include the following design features:

- Bus travel area designed to accommodate the anticipated type of bus for all turning movements;
- Bus loading areas located to reduce conflict between buses and private vehicles;
- Maximum walking distance of 650-ft;
- Bus access points separate from private vehicle access points if demand exceeds 500 all-day spaces;
- Parking placed in the following order with respect to proximity of the bus loading area:
  - Disabled persons;
  - Bicycles;
  - Motorcycles;
  - Vehicles of park and ride users; and
  - Private vehicular parking.
- Ingress and egress located near midblock on collector and local streets; direct access to arterials and freeway ramps should only be used if projected queues do not interfere with functional areas of nearby intersections; at least two ingress/egress points should be provided to the park and ride facility; right and left turn lanes with adequate storage should be added to all ingress/egress locations;
- Parking lanes in the park and ride lot placed approximately 90 degrees to the bus loading area to facilitate safe, convenient walking to buses; and

- Curbs depressed and wheelchair ramps provided where necessary; disabled parking spaces and pedestrian facilities should be in accordance with *Americans with Disabilities Act Accessibility Guidelines* and *Texas Accessibility Standards*.

Combination Park and Pool/Park and Ride Lot

These combination type lots serve the purposes and combine the features of each of the two types of facilities discussed above.

References

Further information on the planning and design of park and ride facilities may be found in the following publications:

- AASHTO's *Policy on Geometric Design of Streets and Highways*;
- AASHTO's *Guide for Park and Ride Facilities;* and
- TCRP Report 155, *Track Design Handbook for Light Rail Transit*, 2nd Edition, Transportation Research Board (TRB)

Authority and Funding

For fringe parking areas within highway right-of-way, projects are generally developed as any other multiple use project. When parking lots are proposed outside of existing or proposed highway right-of-way, commission approval is required.

Park and pool lots are eligible for Federal-aid participation. Projects are usually located within or adjacent to highway right-of-way outside the central business district, but inside the urbanized area, and consistent with the urban transportation planning process. Operation and maintenance responsibilities should be assigned to local transit, government, or other agencies by agreement.

Parking Along Highways and Arterial Streets

This section deals with parking as it pertains to the mainlanes of a controlled access highway, the frontage roads for such a facility, and parking along urban and suburban arterials. Off-street parking facilities provided within highway right-of-way are discussed in the previous section, Fringe Parking Lots. Rest areas as parking facilities are not considered in this section.

Emergency Parking

Parking on and adjacent to the mainlanes of a controlled access highway will not be permitted except for emergency situations. It is of paramount importance that provisions be made for emergency parking. Shoulders of adequate design provide for this required parking space.

Curb Parking

In general, curb parking on urban/suburban arterial streets and frontage roads should be discouraged. Where speed is low and the traffic volumes are well below capacity, curb parking may be permitted. However, at higher speeds and during periods of heavy traffic movement, curb parking is incompatible with arterial street service and should not be permitted. Curb parking reduces capacity and interferes with free flow of adjacent traffic. Elimination of curb parking can increase the capacity of four-to-six lane arterials by 50 to 60 percent.

If curb parking is used on urban/suburban arterials or frontage roads under the conditions stated above, the following design requirements should be met:

- Provide parking lanes only at locations where needed;
- Parallel parking preferred;
- Confine parking lanes to outer side of street or frontage road;
- Require that parking lane widths be 10-ft; and
- Restrict parking a minimum of 20-ft back from the radius of the intersection to allow for sight distance, turning clearance and, if desired, a short right turn lane.

Section 5 — Rumble Strips

Overview

Centerline and shoulder rumble strips are depressed or raised patterns (i.e. profile pavement markings) used to provide auditory and tactile sensations that serve as a warning mechanism when vehicles leave their respective travel lane. Transverse or in-lane rumble strips are placed perpendicular to the direction of vehicular travel and are used in very limited circumstances. The conditions for use of rumble strips are specified in the respective Traffic Safety Division RS standard. Rumble strips have been shown to be a cost-effective countermeasure for reducing the number and severity of roadway departure crashes. As such, rumble strips have been incorporated into the Safety Score Tools developed by TxDOT's Council on System Safety. Additional information on the Safety Score Tools is available on TxDOT's Design Division intranet webpage.

Considerations for Centerline and Shoulder Rumble Strip Placement

Rumble strips shall not be placed on roadways with a posted speed limit of 45 mph or less. For rural high-speed roadways, rumble strips should be installed as part of new construction, reconstruction and overlay projects, unless engineering/safety judgment determine it would be detrimental to do so. Rumble strips are recommended on high-speed urban roadways where significant numbers of crashes by frequency and percentage of total crashes due to motorist inattention have been identified (e.g., opposing direction crashes, run-off-road crashes). Shoulder rumble strips must not be placed across exit or entrance ramps, acceleration and deceleration lanes, crossovers, gore areas, or intersections with other roadways. Depressed rumble strips shall not be placed across bridge decks. If a concrete shoulder will be used in the near future as a permanent travel lane or a travel lane in a work zone, depressed rumble strips should not be used.

Bicyclists Considerations

In all installations, appropriate riding space for bicyclists should be considered. It is preferred to allow at least 5-ft (6-ft or more desired) beyond the rumble strips to the edge of the paved shoulder. On facilities known to have considerable bicycle traffic, consider providing occasional gaps to allow bicyclists to traverse in and out of the shoulder safely. Refer to Chapter 6 Section 4, Bicycle Facilities for additional guidance on the use of rumble strips with bicycles.

See Traffic Safety Division's Rumble Strip Standards (RS standards), and FHWA rumble strip guidance for additional information:
https://safety.fhwa.dot.gov/roadway_dept/pavement/rumble_strips/

Section 6 — Emergency Crossovers

Overview

Emergency crossovers between divided roadways are sometimes necessary for use by official vehicles (e.g., law enforcement, emergency responders, or traffic incident management).

Location

When selecting the location for an emergency crossover, the following guidance should be used:

- In general, emergency crossovers should not be provided on urban roadways due to the close spacing of interchanges.
- Where the spacing of interchanges is greater than approximately 3 miles, an emergency crossover should be considered at a favorable location about halfway between interchanges.
- In no case should an emergency crossover be spaced at less than 1-mile intervals.
- All emergency crossovers should be at least 0.5 mile from any structure that crosses over a roadway and at least 1 mile from any ramp terminal or other access connection, such as those serving safety rest areas.
- Emergency crossovers should be located where adequate stopping sight distance is available and where the median is sufficiently wide to permit an official vehicle to turn between the inner lanes.
- When emergency crossovers are desired on divided facilities with continuous concrete barrier for medians not meeting clear zone requirements of Table 2-12, use TxDOT's Barrier Gate Standard to bridge the opening.
- Emergency crossovers should not be located in curves requiring superelevation, unless engineering judgement determines the location is safe and reasonable for official use.
- Emergency crossovers should also be as inconspicuous to the traveling public as possible. Refer to the TMUTCD for the appropriate regulatory signing of emergency crossovers.

Construction

Location and type of emergency crossovers should be made a part of the PS&E as a contract item and should be installed as such. Refer to Appendix A Section 9, Emergency Crossovers for additional details regarding the design and construction of emergency crossovers.

Section 7 — Minimum Designs for Truck and Bus Turns

Overview

This section contains the following information on minimum designs for truck and bus turns:

- Application;
- Channelization;
- Alternatives to simple curvature;
- Urban intersections;
- Rural intersections; and
- Median U-Turn Movements

Application

Although there are no firm guidelines governing the selection of the type of large vehicle to be used as a design vehicle, there is suggested guidance provided for urban and rural intersections near the end of this section. Factors that influence design vehicle selection are as follows:

- Type and frequency of use by large vehicles;
- Consequences of encroachment into other lanes or the roadside;
- Availability of right-of-way; and
- Functional class of intersecting routes and location (urban versus rural) affect this selection in a general sense. Project-specific traffic data, specifically the frequency of use by the various design vehicle classes, is often the most important consideration in the selection process. The Transportation Planning and Programming Division (TPP) may be contacted to obtain volume data for the various vehicle classes.

Refer to AASHTO's *A Policy on Geometric Design of Highways and Streets* for information on turning paths and turning radii of design vehicles.

Additionally, the use of AutoTURN, AutoTrack, or a similar software should be used to determine the extent of design vehicle encroachment.

Channelization

Where the inner edges of pavement for right turns at intersections are designed to accommodate semi-trailer combinations or where the design permits passenger vehicles to turn at 15-mph or more (i.e., 50-ft or more radius), the pavement area at the intersection may become excessively large for proper control of traffic. In these cases, channelizing islands should be used to more effectively control, direct, and/or divide traffic paths. Physically, islands should be at least 50-ft^2 in urban and 75-ft^2 for rural conditions (100-ft^2 preferable for both) and may be painted or curbed.

Additional guidance for channelization of a right turn slip lane is provided in Appendix D, Right-Turn Slip Lane Design Guidelines.

Alternatives to Simple Curvature

To accommodate the longest vehicles, off-tracking characteristics in combination with the large (simple curve) radius, results in a wide pavement area. In this regard, three-centered compound curves, or offset simple curves in combination with tapers, are preferred since they more closely fit the paths of vehicles. Table 7-2 shows minimum edge of pavement designs for right turns to accommodate various design vehicles for turn angles varying from 60 to 120 degrees.

Table 7-2: Minimum Edge of Pavement Designs for Right Turns for Various Design Vehicles for Turn Angle Varying from 60 to 120 Degrees

Angle of Turn[1] (degrees)	Design Vehicle	Simple Curve Radius (ft.)	Simple Curve Radius with Taper			3-Centered Compound Curve, Symmetric		3-Centered Compound Curve, Asymmetric	
			Radius (ft.)	Offset (ft.)	Taper	Radii (ft.)	Offset(ft.)	Radii (ft.)	Offset (ft.)
60	P	40	-	-	-	-	-	-	-
-	SU	60	-	-	-	-	-	-	-
-	WB-40	90	-	-	-	-	-	-	-
-	WB-62	170	140	4.0	15:1	400-100-400	15.0	110-100-220	10.0-12.5
-	WB-67	200	140	4.5	15:1	400-100-400	8.0	250-125-600	1.0-6.0
75	P	35	25	2.0	10:1	100-25-100	2.0	-	-
-	SU	55	45	2.0	10:1	120-45-120	2.0	-	-
-	WB-40	-	60	2.0	15:1	120-45-120	5.0	120-45-195	2.0-6.5
-	WB-62	-	145	4.0	20:1	440-75-440	15.0	140-100-540	5.0-12.0
-	WB-67	-	145	4.5	20:1	420-75-420	10.0	200-80-600	1.0-10.0
90	P	30	20	2.5	10:1	100-20-100	2.5	-	-
-	SU	50	40	2.0	10:1	120-40-120	2.0	-	-
-	WB-40	-	45	4.0	10:1	120-40-120	5.0	120-40-200	2.0-6.5
-	WB-62	-	120	4.5	30:1	400-70-400	10.0	160-70-360	6.0-10.0
-	WB-67	-	125	4.5	30:1	440-65-440	10.0	200-70-600	1.0-11.0

Table 7-2: Minimum Edge of Pavement Designs for Right Turns for Various Design Vehicles for Turn Angle Varying from 60 to 120 Degrees

Angle of Turn[1] (degrees)	Design Vehicle	Simple Curve Radius (ft.)	Simple Curve Radius with Taper			3-Centered Compound Curve, Symmetric		3-Centered Compound Curve, Asymmetric	
			Radius (ft.)	Offset (ft.)	Taper	Radii (ft.)	Offset(ft.)	Radii (ft.)	Offset (ft.)
105	P	-	20	2.5	-	100-20-100	2.5	-	-
-	SU	-	35	3.0	-	100-35-100	3.0	-	-
-	WB-40	-	40	4.0	-	100-35-100	5.0	100-55-200	2.0-8.0
-	WB-62	-	115	3.0	15:1	520-50-520	15.0	360-75-600	4.0-10.5
-	WB-67	-	115	3.0	15:1	500-50-500	13.0	200-65-600	1.0-11.0
120	P	-	20	2.0	-	100-20-100	2.0	-	-
-	SU	-	30	3.0	-	100-30-100	3.0	-	-
-	WB-40	-	35	5.0	-	120-30-120	6.0	100-30-180	2.0-9.0
-	WB-62	-	100	5.0	15:1	520-70-520	10.0	80-55-520	24.0-17.0
-	WB-67	-	105	5.2	15:1	550-45-550	15.0	200-60-600	2.0-12.5

Notes:
1. "Angle of Turn" is the angle through which a vehicle travels in making a turn. It is measured from the extension of the tangent on which a vehicle approaches to the corresponding tangent on the intersecting road to which a vehicle turns. It is the same angle that is commonly called the delta angle in surveying terminology.

Figure 7-46 shows sample alternatives to simple curvature edge of pavement geometry for a 90-degree turn using a WB-62 design vehicle. Although not shown in this figure, a radius of 100-ft without channelizing island would be necessary to accommodate the wide, off-tracking path of a WB-62 without undesirable encroachment. A geometric design of this sort is undesirable because such a wide pavement may confuse drivers and limit effective space for traffic control devices.

Vehicle swept path analysis software (e.g. AutoTURN or similar software) can also be used to analyze turning movements through intersections to optimize edge of pavement geometry and ensure the design vehicle can adequately maneuver the intersection.

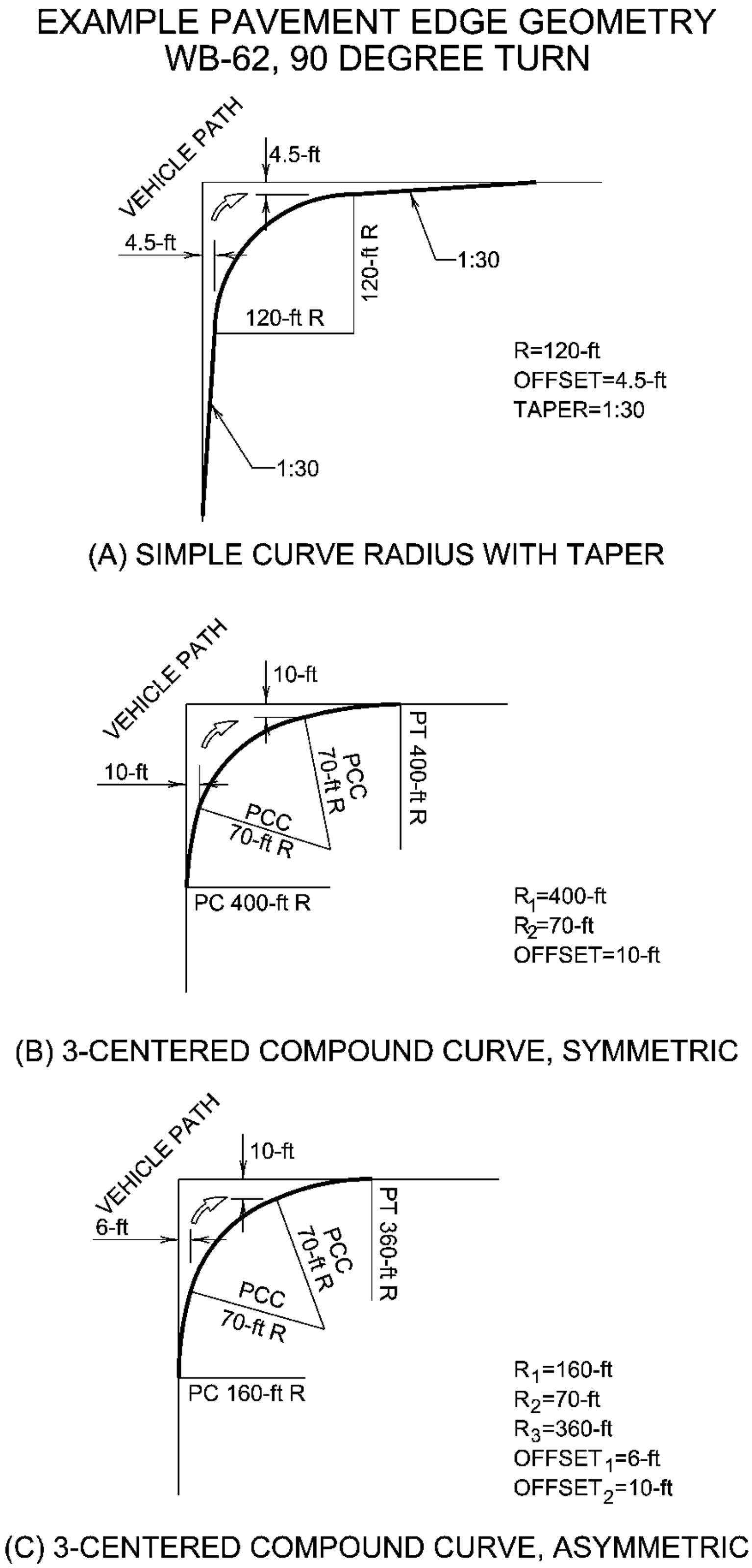


Figure 7-46. Example of Pavement Edge Geometry

Urban Intersections

Corner radii at intersections on arterial streets should satisfy the requirements of the drivers using them to the extent practical and in consideration of the amount of right-of-way available, the angle of the intersection, numbers of and space for pedestrians, width and number of lanes on the intersecting streets, and amounts of speed reductions. The following summary is offered as a guide:

- Radii of 15-ft to 25-ft are adequate for passenger vehicles. These radii may be provided at minor cross streets where there is little occasion for trucks to turn or at major intersections where there are parking lanes. Where the street has sufficient capacity to retain the curb lane as a parking lane for the foreseeable future, parking should be restricted for appropriate distances from the crossing.
- Radii of 25-ft or more at minor cross streets should be provided on new construction and on reconstruction where space permits.
- Radii of 30-ft or more at major cross streets should be provided where feasible so that an occasional truck can turn without too much encroachment.
- Radii of 40-ft or more, and preferably 3-centered compound curves or simple curves with tapers to fit the paths of appropriate design vehicles, should be provided where large truck combinations and buses turn frequently. Larger radii are also desirable where speed reductions would cause problems.
- Radii of 75-ft or more should be provided for arterial-arterial urban intersections where frequent use a WB-62 or larger design vehicle is anticipated.
- Radii dimensions should be coordinated with crosswalk distances or special designs to make crosswalks safe for all pedestrians.

Where other types of truck combinations are used as the design vehicle, pavement edge geometry as shown in Table 7-2 and Figure 7-46 permit the use of lesser radii. An operational measure that appears promising is to provide guidance in the form of edge lines to accommodate the turning paths of passenger cars, while providing sufficient paved area beyond the edge lines to accommodate the turning path of an occasional large vehicle.

Rural Intersections

In rural areas space is generally more available and speeds higher. These factors suggest more liberal designs for truck turning even when the frequency of long vehicles may not be as great as in urban areas.

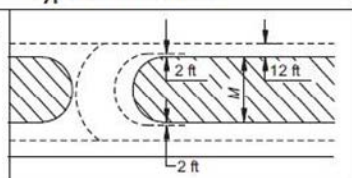
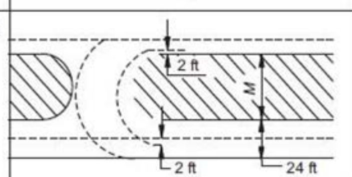
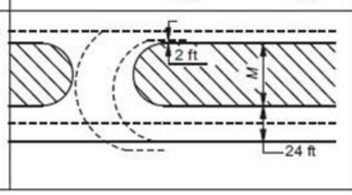
In the design of highway intersections with other (non-highway system) public roads, long vehicles are generally infrequent users. Minimally, the SU, or on some occasions the WB-40, design vehicle is appropriate for use unless special circumstances (location of a truck stop or terminal) influence the frequency of use by certain vehicle classes.

For arterial intersections with collectors, the WB-40 design vehicle is generally appropriate and the WB-62 should be used where specific circumstances warrant.

For arterial-arterial intersections, use by the WB-62 or larger design vehicle should be anticipated within project life. For turning roadway widths to be reasonable, a design radius of 75-ft or more is required. Where circumstances at a particular rural arterial-arterial intersection precludes the use of the WB-62 or larger design vehicle, the WB-40 may be used. Refer to Table 7-2 for radii information with respect to the design vehicle.

Median U-Turn Movements

In some instances, median openings can be used to facilitate U-turns. Figure 7-47 provides minimum median widths “m” (ft) to complete a U-turn movement. For instances where the U-turn can't be completed, a loon may be used to provide additional space. Refer to Appendix E, Section 4, Median U-Turn Intersection (MUT) for additional information on loon design.

U.S. Customary								
		M—Minimum Width of Median (m) for Design Vehicle						
		P	WB-40	SU-30	BUS	SU-40	WB-62	WB-67
		Length of Design Vehicle (ft)						
Type of Maneuver		19	50	30	40	40	63	68
Inner Lane to Inner Lane		30	61	63	63	76	69	69
Inner Lane to Outer Lane		18	49	51	51	64	57	57
Inner Lane to Shoulder		8	39	41	41	54	47	47

Source: FHWA MUT Informational Guide - Exhibit 7-12

Figure 7-47. AASHTO Minimum Median Widths for U-Turn Crossovers

Chapter 8 — Mobility Corridor (5 R) Design Criteria

Contents:

Section 1 — Overview

Introduction

Mobility corridors are intended to regenerate, or produce new, long-term transportation opportunities. These transportation opportunities may include multiple modes such as rail, utilities, freight, and passenger characteristics. These modes may occur within a single corridor alignment or the modes may be separated for some intervals. This chapter is intended to provide design guidance on the roadway aspects of these mobility corridors. This guidance can be expected to be updated as experience is gained in the planning, design, construction, and operations of these transportation facilities.

The primary focus of these corridors is mobility. The roadway portions of a mobility corridor facility are intended for long-distance travel and will therefore be controlled in terms of access. Access will be limited to public roadways via ramp connections. Access will not be allowed along the ramps connecting to the mobility corridor.

Since these corridors are intended for mobility, the design speeds presented in this chapter are between 85-mph to 100-mph. Because mobility corridors may be generated or regenerated, this design criteria may be applied when planning new facilities or reconstructing existing corridors. While higher operating speeds may not be appropriate in all instances (such as densely developed urban areas), these higher design speeds can be applied, and should be considered, whenever prudent.

With respect to facilities that one day could be part of a major corridor, particularly new location routes, it is strongly recommended that these facilities be initially designed to accommodate a 100-mph design speed. Even though the facility may initially be posted for an 85-mph speed, the higher speed design criteria will allow the greatest flexibility, both in the roadway portion as well as for other transportation modes within the right of way, in terms of maximizing the future use of the corridor.

This does not mean that all projects must be over-designed. If it is determined through the project development process that substantial, adverse and unavoidable social, economic and environmental impacts will occur, then different design criteria may be appropriate. Contact the Environmental Affairs Division and the Right-of-Way Division with questions about environmental and right-of-way impacts while planning for higher design speeds.

As always, the potential long-term use and growth of the system must be considered and appropriate planning and engineering principles must be applied. Again, these mobility corridors are not primarily intended for local travel.

5R projects must be assessed to determine if bicycle accommodations are required per Chapter 6, Section 4, Bicycle Facilities; if bicycle facilities are provided, they must meet the additional requirements specified in Chapter 6, Section 4.

Section 2 — Roadway Design Criteria

Overview

This section discusses the features and design criteria for the roadway portion of mobility corridors and includes the following subsections:

- Lane Width and Number;
- Shoulders;
- Pavement Cross Slope;
- Vertical Clearances at Structures;
- Stopping Sight Distance;
- Grades;
- Horizontal Alignment;
- Superelevation;
- Superelevation Transition; and
- Vertical Curves.

Departure from these guidelines are governed in Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight Network (THFN) Design Deviations, Chapter 1.

Lane Width and Number

The usual and minimum lane width is 13-ft. The number of lanes required to accommodate the anticipated design year traffic is determined by the level of service evaluation as discussed in TRB's *Highway Capacity Manual*.

Shoulders

The minimum shoulder width is 12-ft. This width applies to both inside and outside shoulders, regardless of the number of main lanes. Shoulders must be continuously surfaced and be maintained.

Pavement Cross Slope

Multilane divided pavements must be inclined in the same direction. The recommended pavement cross slope is 2 percent. Shoulders should be sloped sufficiently to drain surface water but not to an extent that safety concerns are created for vehicular use. To facilitate pavement drainage, highways with three or more lanes inclined in the same direction should have an increasing cross slope as the distance from the crown line increases. In these cases, the first two lanes adjacent to the crown line may be sloped flatter than normal-typically at 1.5 percent but not less than 1 percent. The cross slope of each successive pair of lanes (or single lane if it is the outside lane) outward from the crown should be increased by 0.5 to 1 percent from the cross slope of the adjacent lane. A cross slope should not exceed 4 percent on a tangent alignment unless there are three or more lanes in one direction of travel. Bridge structures with three or more lanes in one direction should maintain a constant slope of 2.5 percent, transitioning before and after the bridge accordingly.

Vertical Clearances at Structures

The minimum vertical clearances at structures are shown in Table 2-11.

Stopping Sight Distance

Stopping sight distance (SSD) for these facilities is calculated using the same methodology described in Chapter 2, Section 3, Sight Distance. The key variables that affect the calculation of SSD are brake reaction time and deceleration rate. The calculated and design stopping sight distances are shown in Table 8-1. Significant downgrades may affect stopping sight distances.

Table 8-1: Stopping Sight Distances for 5R Projects

Design Speed (mph)	Brake reaction distance (ft)	Braking distance on level (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
85	313.5	693.5	1,007	1,010
90	330.8	777.5	1,108.2	1,110
95	349.1	866.2	1,215.4	1,220
100	367.5	959.8	1,327.3	1,330
Note: Brake reaction distance predicated on a time of 2.5-sec; deceleration rate 11.2-ft/sec.				

Grades

Undesirable speed differentials between vehicle types suggest that limiting the rate and length of the grades be considered. Passenger vehicles are not significantly affected by grades as steep as 3 percent, regardless of initial speed. Grades above 2 percent may affect truck traffic depending on length of grade.

Table 8-2 summarizes the maximum grade controls in terms of design speed.

Table 8-2: Maximum Grades for 5R Projects

Terrain	Design Speed (mph)			
	85	90	95	100
Level	2-3	2-3	2-3	2-3
Rolling	4	4	4	4

Horizontal Alignment

Table 8-3 shows the maximum allowable side friction factors and assumed running speeds for design speeds from 85-mph to 100-mph. The maximum side friction force is used for full superelevation in conditions where limited space places constraints on the horizontal geometry and should be avoided.

Table 8-3: Side Friction Factors and Running Speeds for Horizontal Curves

Design Speed (mph)	Maximum Allowable Friction Factor	Running Speed (mph)
85	0.07	67
90	0.06	70
95	0.05	75[1]
100	0.04	82[1]
Note: 1. Values adjusted up to accommodate application of AASHTO Method 5 calculations.		

Superelevation

Table 8-4 and Table 8-5 show minimum superelevation rates of various radii and design speeds for an e_{max} of 6 percent and 8 percent, respectively. For multi-lane facilities, particularly where wide medians are used, the radius applies to the innermost travel lane.

Table 8-4: Minimum Radii and Superelevation Rates[1] for Mobility Corridors, e_{max} = 6%

Superelevation Rate, e (%)	Radius, R (ft)			
	85-mph	90-mph	95-mph	100-mph
NC[2,4]	29,310	32,190	37,140	44,420

Table 8-4: Minimum Radii and Superelevation Rates[1] for Mobility Corridors, e_{max} = 6%

Superelevation Rate, e (%)	Radius, R (ft)			
	85-mph	**90-mph**	**95-mph**	**100-mph**
RC[3,4]	14,290	15,850	18,350	22,010
2.2	12,930	14,360	16,650	19,970
2.4	11,790	13,120	15,230	18,270
2.6	10,830	12,070	14,020	16,830
2.8	10,000	11,170	12,990	15,600
3.0	9,290	10,400	12,100	14,530
3.2	8,660	9,710	11,320	13,590
3.4	8,110	9,110	10,630	12,770
3.6	7,610	8,580	10,010	12,040
3.8	7,170	8,100	9,460	11,380
4.0	6,770	7,660	8,970	10,790
4.2	6,410	7,270	8,520	10,250
4.4	6,080	6,920	8,110	9,770
4.6	5,780	6,590	7,740	9,330
4.8	5,510	6,300	7,400	8,920
5.0	5,260	6,020	7,090	8,540
5.2	5,020	5,770	6,800	8,200
5.4	4,790	5,530	6,530	7,880
5.6	4,550	5,310	6,280	7,580
5.8	4,260	5,040	6,020	7,280
6.0	3,710	4,500	5,470	6,670

Notes:

1. Computed using Superelevation Distribution Method 5. See AASHTO's A Policy on Geometric Design of Highways and Streets for the different types of Superelevation Distribution Methods.
2. a) The term "NC" (normal crown) represents an equal downward cross-slope, typically 2%, on each side of the axis of rotation.
   b) The minimum curve radii for normal crown are suitable up to 3.0%.
   c) 3.0% normal crown should only be used when 3 or more lanes are sloped in the same direction.
   d) 1.5% or flatter normal crown should only be used for the design of special circumstance, such as table-topping intersections, or the evaluation of existing conditions.
3. The term "RC" (reverse crown) represents a curve where the downward, or adverse, cross-slope should be removed by superelevating the entire roadway at the normal cross-slope rate.
4. For curve radii falling between normal crown and reverse crown, a superelevation rate equal to the normal crown should typically be used.

Table 8-5: Minimum Radii and Superelevation Rates[1] for Mobility Corridors, e_{max} = 8%

Superelevation Rate, e (%)	Radius, R (ft)			
	85-mph	90-mph	95-mph	100-mph
NC[2,4]	29,700	32,580	37,530	44,870
RC[3,4]	14,700	16,220	18,730	22,400
2.2	13,330	14,740	17,020	20,360
2.4	12,200	13,500	15,600	18,660
2.6	11,240	12,450	14,400	17,220
2.8	10,420	11,550	13,370	15,990
3.0	9,700	10,780	12,470	14,920
3.2	9,080	10,100	11,690	13,990
3.4	8,530	9,490	11,000	13,160
3.6	8,040	8,960	10,390	12,430
3.8	7,600	8,480	9,840	11,770
4.0	7,210	8,050	9,350	11,180
4.2	6,850	7,660	8,900	10,650
4.4	6,530	7,310	8,490	10,160
4.6	6,230	6,990	8,120	9,720
4.8	5,960	6,690	7,780	9,320
5.0	5,710	6,420	7,470	8,940
5.2	5,480	6,170	7,180	8,600
5.4	5,260	5,930	6,910	8,280
5.6	5,060	5,720	6,670	7,980
5.8	4,880	5,520	6,440	7,700
6.0	4,710	5,330	6,220	7,450
6.2	4,550	5,150	6,020	7,210
6.4	4,390	4,990	5,830	6,980
6.6	4,250	4,830	5,650	6,770
6.8	4,120	4,690	5,490	6,570
7.0	3,990	4,550	5,330	6,380
7.2	3,870	4,420	5,180	6,200
7.4	3,760	4,300	5,040	6,030
7.6	3,640	4,180	4,900	5,870
7.8	3,510	4,070	4,780	5,720
8.0	3,210	3,860	4,630	5,560

Table 8-5: Minimum Radii and Superelevation Rates[1] for Mobility Corridors, e_{max} = 8%

Superelevation Rate, e (%)	Radius, R (ft)			
	85-mph	**90-mph**	**95-mph**	**100-mph**

Notes:

1. Computed using Superelevation Distribution Method 5. See AASHTO's A Policy on Geometric Design of Highways and Streets for the different types of Superelevation Distribution Methods.
2. a) The term "NC" (normal crown) represents an equal downward cross-slope, typically 2%, on each side of the axis of rotation.
   b) The minimum curve radii for normal crown are suitable up to 3.0%.
   c) 3.0% normal crown should only be used when 3 or more lanes are sloped in the same direction.
   d) 1.5% or flatter normal crown should only be used for the design of special circumstance, such as table-topping intersections, or the evaluation of existing conditions.
3. The term "RC" (reverse crown) represents a curve where the downward, or adverse, cross-slope should be removed by superelevating the entire roadway at the normal cross slope-rate.
4. For curve radii falling between normal crown and reverse crown, a superelevation rate equal to the normal crown should typically be used.

Superelevation Transition

Desirable design values for length of superelevation transition are based on a given maximum relative gradient between profiles of the edge of traveled way and the axis of rotation. Table 8-6 shows recommended maximum relative gradient values. Transition length on this basis is directly proportional to the total superelevation, which is the product of the lane width and the change in the cross slope. For superelevation on bridge structures, it is preferred to begin/end superelevation transition at a bridge bent line.

Table 8-6: Maximum Relative Gradient for Superelevation Transition

Design Speed (mph)	Maximum Relative Gradient[1] (%)	Equivalent Maximum Relative Slope (V:H)
85-100	0.50	1:200

Note:

1. Maximum relative gradient for profile between edge of traveled way and axis of rotation.

Vertical Curves

Vertical curves create a gradual transition between different grades which is essential for the safe and efficient operation of a roadway. The lengths of both crest and sag vertical curves are controlled by the available sight distance. Vertical curves are required for all grade breaks.

Minimum K-values are calculated using the same equations as in Chapter 2, Section 6.

Design Ks for both crest and sag vertical curves are shown on Table 8-7.

Table 8-7: Minimum Design K for Crest and Sag Vertical Curves

Design Speed (mph)	Stopping Sight Distance (ft)	Crest Vertical Curves (K)	Sag Vertical Curves (K)
85	1,010	473	260
90	1,110	571	288
95	1,220	690	319
100	1,330	820	350

The length of a sag vertical curve that satisfies the driver comfort criteria is 60 percent of the sag vertical curve length required by the sight distance control. Driver comfort control should be reserved for special use and where continuous lighting systems are in place.

Section 3 — Roadside Design Criteria

Clear Zone

The clear zone distances are shown in Table 8-8.

Table 8-8: 5R Clear Zones

Design Speed (mph)	Clear Zone Distance (ft)
85	80
90	80
95	90
100	100

Slopes

For safety, it is desirable to design relatively flat areas adjacent to the travelway so that out-of-control vehicles are more likely to recover or make a controlled deceleration. Design guide values for the selection of earth fill slope rates in relation to height of fill are shown in Table 8-9. Particularly difficult terrain may require deviation from these general guide values. Where conditions are favorable, it is desirable to use flatter slopes to enhance roadside safety.

Table 8-9: Earth Fill Slope Rates

Height of Fill (ft)	Desirable Slope Rate[1] (V:H)	
	Terrain	
	Level	Rolling
0-5	1:8	1:6
5 and over	1:6	1:6

Note:
1. Slope rates should be flatter where practical. Deviation may be permitted for particularly difficult terrain conditions.

The slope adjacent to the shoulder is called the front slope. Ideally, the front slope should be 1V:8H or flatter, although steeper slopes are acceptable in some locations.

The back slope should typically be 1V:6H or flatter. However, the slope ratio of the back slope may vary depending upon the geologic formation encountered. For example, where the roadway alignment traverses through a rock formation area, back slopes are typically much steeper.

The intersections of slope planes in the highway cross section should be well rounded for added safety and increased stability of out-of-control vehicles. Where barrier is placed on side slopes, the area between the roadway and barrier must be sloped at 1V:10H or flatter.

Medians

The median width is the distance between the inside edge of travel lanes of opposing traffic. Median barriers must be installed when the median widths are less than those shown in Table 8-8.

Section 4 — Ramps and Direct Connectors

Overview

Ramps and direct connectors are designed to the same criteria.

This section discusses ramps and direct connectors and includes information on the following topics:

- Design Speed;
- Lane and Shoulder Widths;
- Acceleration and Deceleration Lengths;
- Distance Between Successive Ramps;
- Grades and Profiles; and
- Cross Section and Cross Slopes

Design Speed

Similar to facilities with design speeds of 80-mph or less, ramps on these facilities must also have a relationship between the ramp design speed and the mainlane design speed. All ramps and direct connectors must be designed to enable vehicles to leave and enter the travel way of the highway at 85 percent (desirable) to 70 percent (usual minimum) of the highway design speed, rounded up to the nearest 5-mph increment, and limiting the speed differential to 10-mph on the upper range and 20-mph for the mid-range. Every effort should be made to meet the desirable ramp/connector design speed.

Table 8-10 shows the values for ramp/connector design speeds.

Table 8-10: Guide Values for Ramp/Connector Design Speed as Related to Highway Design Speed[1]

Ramp/Connector Design Speed[2] (mph)	Highway Design Speed (mph)			
	85	90	95	100
Upper Range (85%)	75	80	85	90
Mid-Range (70%)	65	70	75	80
Lower Range (50%)	55	60	65	70

Table 8-10: Guide Values for Ramp/Connector Design Speed as Related to Highway Design Speed[1]

Ramp/Connector Design Speed[2] (mph)	Highway Design Speed (mph)			
	85	90	95	100

Notes:
1. Values determined by the percentage of the highway design speed or the max difference between highway and ramp design speed (10 mph for upper, 20 mph for mid, 30 mph for lower), whichever is higher.
2. For corresponding minimum radius, see Table 2-4, Table 2-5, Table 8-4 and Table 8-5.

Lane and Shoulder Widths

Ramp and direct connector shoulder widths (inside and outside) and lane widths are shown in Table 8-11.

Table 8-11: Ramp and Direct Connection Widths

Number of Lanes	Inside Shoulder Width (ft)	Outside Shoulder Width[1] (ft)	Traffic Lanes (ft)
1	8	10	14
2	4	10	26 (14/12)

Note:
1. If sight distance restrictions are present due to horizontal curvature, the shoulder width on the inside of the curve may be increased to 10-ft and the shoulder width on the outside of the curve decreased to 8-ft (one lane) or 4-ft (two lane).

Acceleration and Deceleration Lengths

Table 8-12 provides design criteria for exit ramp deceleration and taper lengths. Adjustment factors for grade effects are independent of highway design speed, therefore use Table 3-14 for deceleration length adjustment factors.

Table 8-12: Minimum Deceleration Lane Lengths for Exit Ramp Speed Change Lanes with Flat Grades of Less than 3 Percent

	Design Speed of Controlling Feature on Ramp, V' (mph)							
	55	60	65	70	75	80	85	90
Highway Design Speed, V (mph)	Deceleration Lane Length[1], La (ft)							
85	445	385	330	275	220	125	-	-

Table 8-12: Minimum Deceleration Lane Lengths for Exit Ramp Speed Change Lanes with Flat Grades of Less than 3 Percent

	Design Speed of Controlling Feature on Ramp, V' (mph)							
	55	**60**	**65**	**70**	**75**	**80**	**85**	**90**
Highway Design Speed, V (mph)	**Deceleration Lane Length[1], La (ft)**							
90	495	445	395	350	305	235	135	-
95	535	490	445	410	380	315	240	135
100	575	535	495	465	445	395	330	250

Note:
1. Where providing desirable deceleration length is impractical, it is acceptable to allow for a moderate amount of deceleration (10 mph) within the through lanes prior to the beginning of the exit ramp.

Table 8-13 provides design criteria for entrance ramp acceleration and taper lengths; adjustment factors for grade effects are shown in Table 8-14.

Table 8-13: Lengths of Entrance Ramp Speed Change Lanes

	Design Speed of Controlling Feature on Ramp, V' (mph)							
	55	**60**	**65**	**70**	**75**	**80**	**85**	**90**
Highway Design Speed, V (mph)	**Deceleration Lane Length, La (ft)**							
85	875	620	375	110	-	-	-	-
90	1095	850	595	355	-	-	-	-
95	1340	1110	850	635	270	-	-	-
100	1620	1405	1135	960	580	140	-	-

Table 8-14: Speed Change Lane Adjustment Factors as a Function of a Grade

Highway Design Speed, V (mph)	**Ratio of Length on Grade to Length on Level[1]**			
	3 to 4 % Upgrade	**3 to 4 % Downgrade**	**5 to 6 % Upgrade**	**5 to 6 % Downgrade**
All	**0.90**	**1.20**	**0.80**	**1.35**
Acceleration Lanes				

Table 8-14: Speed Change Lane Adjustment Factors as a Function of a Grade

Highway Design Speed, V (mph)	Ratio of Length on Grade for Design Speed (mph) of Turning Roadway Curve[1]							
	20	25	30	35	40	45	50	All Speeds
	3 to 4% Upgrade							3 to 4%
85	1.62	1.69	1.75	1.80	1.89	1.99	2.10	0.56
90	1.66	1.73	1.80	1.86	1.96	2.08	2.20	0.55
95	1.71	1.78	1.85	1.92	2.03	2.17	2.30	0.54
100	1.75	1.83	1.90	1.98	2.10	2.26	2.40	0.52
	5 to 6 % Upgrade							5 to 6 %
85	2.39	2.51	2.64	2.94	3.15	3.73	4.28	0.46
90	2.50	2.64	2.77	3.10	3.33	4.00	4.65	0.45
95	2.62	2.76	2.91	3.27	3.51	4.26	5.03	0.44
100	2.74	2.89	3.04	3.43	3.69	4.53	5.40	0.42

Note:
1. Ratio in this table multiplied by length of deceleration or acceleration distances in Table 8-13 and Table 3-14 gives length of deceleration/acceleration distance on grade.

Distance Between Successive Ramps

The minimum acceptable distance between ramps is dependent upon the merge, diverge, and weaving operations that take place between ramps and TRB's *Highway Capacity Manual* must be used for analysis of these requirements. Several iterations of the analysis may be required to determine these lengths at the higher design speeds. The distances required for adequate signing must also be considered.

Grades and Profiles

Grades and profiles are associated with the design speed selected for the ramp. Design criteria for design speeds less than 85-mph can be found in Chapter 2 Table 2-9.

Cross Section and Cross Slopes

The cross slope for ramp tangent sections should be similar to the cross slope used on the mainlanes of the roadway. The cross slope on the ramp should be sloped in the same direction across the entire ramp. The cross slope used will depend on the pavement type and other drainage considerations.

Appendix A — Longitudinal Barriers

Contents:

Section 1 — Overview

Introduction

The objectives of this appendix are to make available data and guidelines for the use of roadside and median traffic barriers in a consolidated and understandable form. These guidelines should be supplemented by sound engineering judgment.

The area adjacent to the traveled way plays an important role in the safe operation of a facility. Crash statistics show that a significant portion of crashes on rural roads are the single vehicle, run-off-the-road type. Provision of an obstacle-free zone and the effective use of barriers to shield obstacles that cannot otherwise be removed or safety treated are important considerations for enhancing safety performance.

Section 2 — Barrier Need

Overview

Traffic barriers are considered only when the obstacle is less forgiving than striking the barrier itself.

Should a roadside obstacle exist, treatment should be considered in the following priority:

1. Remove obstacle.
2. Redesign the obstacle so it can be safely traversed.
3. Relocate the obstacle to a point where it is less likely to be struck.
4. Reduce impact severity by using an appropriate breakaway device.
5. Shield the obstacle with a longitudinal traffic barrier designed for redirection or use a crash cushion.
6. Delineate the obstacle if the above alternatives are not appropriate.

Types of Barrier

Rigid

Common rigid barriers used by TxDOT are constant single-slope barriers and F-shape barriers that are cast in place or precast. Concrete barriers placed in situ or keyed into the roadway pavement are considered rigid barriers. Concrete barriers are placed primarily where little movement of the barrier can be tolerated and typically do not deform when impacted. For concrete barriers that are anchored, between 0 and 1-ft of deflection is anticipated for a TL-3 (high-speed) pickup truck impact. Where considerable truck traffic exists or is anticipated, a TL-4 rated barrier should be considered. (Minimum height of a TL-4 barrier is 36 inches). Concrete barriers are typically used at narrow medians. The exposed ends of the barrier need to be protected by an attenuator.

Semi-Rigid

Semi-rigid barriers commonly used by TxDOT include metal beam guard fence and both pinned and unanchored precast concrete barriers. Semi-rigid barriers have an expected deflection between 26 and 60 inches if impacted by a MASH TL-3 pickup truck under MASH test conditions. Metal beam guard fence is the most commonly used barrier by TxDOT. The current height tolerance for a new installation of a (MGS) system is 31 inches plus or minus 1 inch measured from the road surface to the top of w-beam rail. Metal beam guard fence is used primarily to shield roadside obstacles, such as slopes, utility poles, or trees. Precast concrete barriers can be used for temporary or permanent installations. Precast concrete barrier is most commonly used in work zones to shield personnel from traffic.

Flexible

High-tension cable barriers are the most commonly used flexible barriers. A cable barrier is sometimes referred to as a wire rope safety barrier. It consists of high-tension steel cables mounted on weak posts with a post foundation and typical anchor terminal. Cable barriers are used as median barriers to reduce median crossover vehicle encroachments.

Additional guidance for each barrier type is provided in subsequent sections.

Barrier Transitions

When transitioning from one type of barrier to another with significant differences in deflection or working width it is necessary to provide a transition between the two barrier types to gradually increase stiffness to reduce the risk of pocketing and/or snagging. TxDOT has several standards for transitions from Rigid to Semi-rigid (metal beam guard fence) barriers available on the Design Division Roadway standards webpage. The purpose of all these transitions is to gradually decrease the expected deflection to safely redirect an errant vehicle.

Special Barrier Applications

In the application of traffic control plans, situations may arise where transitions are needed from a concrete barrier that has less anticipated deflection to barriers where more deflection is anticipated. The continuum for deflection from less deflection to greater deflection is cast-in-place, precast pinned, and precast non-pinned respectively. Additionally, a temporary barrier run of 120-ft or less would generally be considered a short run and greater deflection would be anticipated. See the Guidebook for Use of Pinned-Down Temporary Concrete Barriers in Limited Space Applications - Roadside Safety Pooled Fund (2016) for additional guidance, or contact the Roadway Design Section of Design Division.

Applications

The three basic types of obstacles that are commonly shielded using roadside barriers are as follows:

- Slopes, lateral drop-offs, or terrain features;
- Bridge ends and the areas alongside bridges; and
- Other roadside obstacles that cannot be eliminated, made breakaway or otherwise traversable, or relocated.

Table A-1 shows a summary of roadside features that are commonly shielded with guard fence. The abbreviation cz means within clear zone.

Table A-1: General Applications of Conditions for Roadside Barriers

Roadside Feature	Applications
Terrain Features:	
◆ Steep Embankment Slope	◆ cz[1], See Figure A-1
◆ Rough Rock Cut	◆ cz
◆ Boulders	◆ cz, diameter exceeds 6-in
◆ Water Body (e.g. Ponds, Detention/Retention Ponds, Lakes, and Rivers/Streams)	◆ cz, depth exceeds 2-ft, permanent
◆ Lateral Drop-off (e.g. Borrow Pits)	◆ cz & steeper than 1V:1H and depth exceeds 2-ft
◆ Side Ditches	◆ cz & unsafe cross section[2]
Bridges:	
◆ Parapet Wall/Wingwall/Bridge Rail End	◆ approaching traffic, or within cz of opposing traffic
◆ Area Alongside Bridges	◆ approaching traffic, or within cz of opposing traffic
Roadside Obstacles:	
◆ Trees	◆ cz & diameter exceeds 4-in
◆ Culvert Headwall[3]	◆ cz & size of opening exceeds 3-ft (without safety grates only)
◆ Wood Poles, Posts	◆ cz & cross section/area exceeds 50 in^2
◆ Bridge Piers, Abutments at Underpasses	◆ cz
◆ Retaining Walls, Noise Wall	◆ cz & not parallel to travelway

Notes:
1. cz – Within clear zone for highway class and traffic volume conditions.
2. For preferred ditch cross sections, see Chapter 2 Section 8, Side Ditches.
3. For specific 4R requirements see Chapter 2 Section 7, Cross Sectional Elements; for 3R see Chapter 4 Section 3, Designing for Safety.

When evaluating roadside design, consideration should be given to the possible removal of metal beam guard fence if it is no longer needed; providing cross sections that make metal beam guard fence unnecessary is desirable. This includes providing a sufficient clear zone recovery area and re-grading ditches to the extent possible in lieu of metal beam guard fence.

Gateway Monuments are considered discretionary items which means they are not necessary for the safety, maintenance, and operation of the roadway. Gateway Monuments, their foundations and any of their associated appurtenances proposed on the roadside of a facility must not be positioned within the clear recovery zone, and it is desirable that any monuments, their foundations and associated appurtenances be placed at least 10-ft beyond the clear zone. If this cannot be achieved, if a more desirable alternative location is not available, and if the monument must be placed within the clear zone, it must be shielded with an appropriate crashworthy device consistent with TxDOT Roadway standards and applications.

The combination of embankment height and side slope rate may indicate barrier protection consideration as shown in Figure A-1. For low fill heights, a more abrupt slope rate is tolerable than at high fill heights. Because steeper than 1V:4H side slopes provide little opportunity for drivers to redirect vehicles at high speeds, in the absence of guard fence, a 10-ft area free of obstructions should be provided by the designer beyond the toe of slope for steeper slopes.

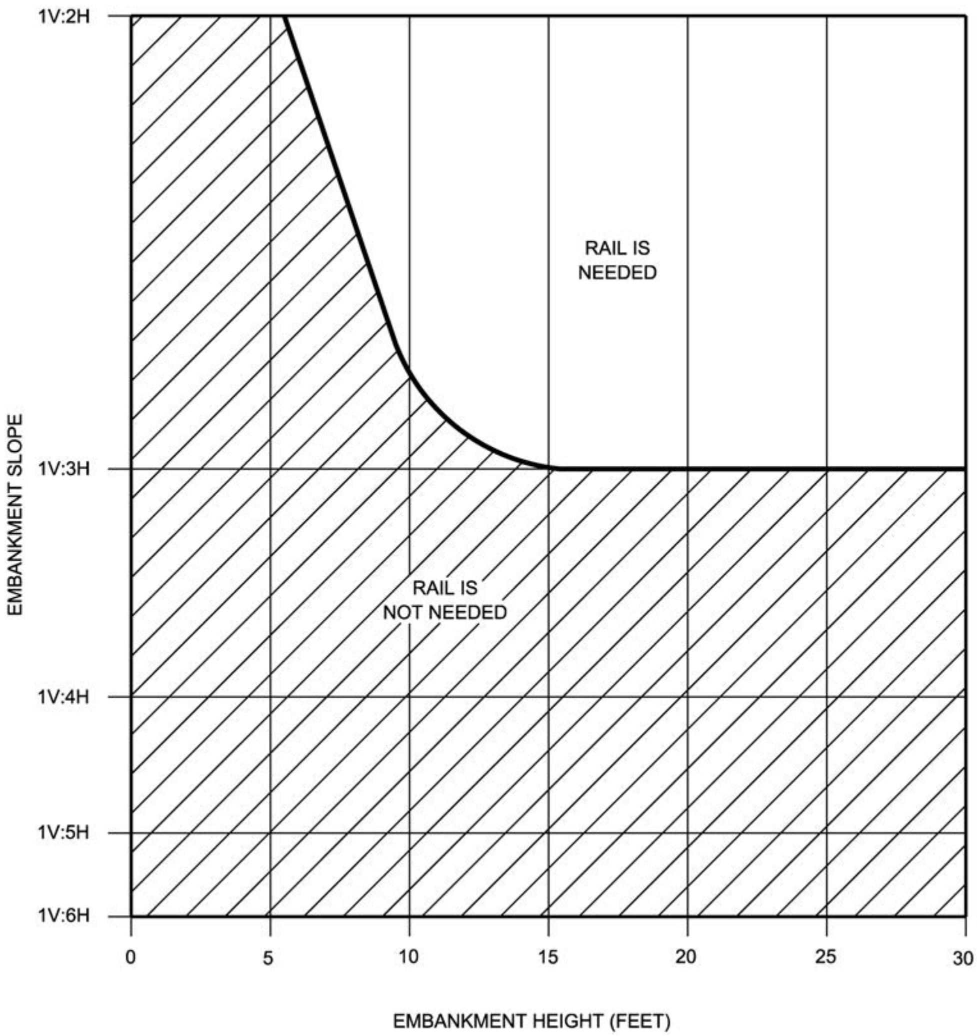


Figure A-1. Guide for Use of Guard Fence for Embankment Heights and Slopes

Working Width

Working width is the distance between the traffic face of the barrier before the impact and the maximum lateral position of any major part of the system or vehicle after the impact. (see Figure A-2). Working width is related to deflection, but working width takes into account the lateral position of the vehicle. Working width should be considered when placing any longitudinal barrier.

Figure A-2. Working Width

Section 3 — Structural Considerations of Guard Fence

Overview

Post spacing, rail shape and thickness, rail height, splice strength and location, post embedment, and rail anchorage are all important factors that influence the structural integrity of guard fence.

Post Spacing, Embedment, and Lateral Support

Typical post spacing is 6-ft 3-in for guard fence. Where guard fence is to be placed at or near the shoulder edge, it is recommended that the cross slope of the shoulder be projected, typically 2-ft beyond the back of the post location as shown in Figure A-3, to provide lateral support for the posts. Locating the roadway cross slope/side slope hinge point behind the rail also provides a platform that increases vehicular stability in the event of impacts that straddle the end section.

Embedment depth is shown on the standard detail sheet for both timber and steel posts.

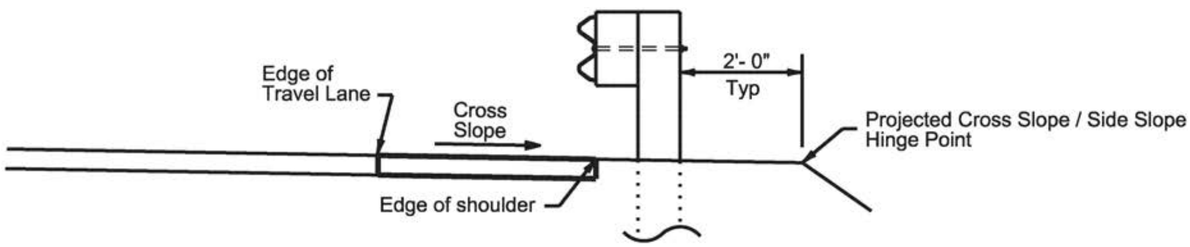


Figure A-3. Lateral Support to Accommodate Guard Fence

Rail Element

Guard fence is fabricated in a deep beam shape to provide for bending strength. Nominal thickness of the rail is 10 or 12 gauge. End treatments, wingwalls, retaining walls, etc. provide firm rail anchorage. With full splice connections, the anchored rail has sufficient tensile and flexural strength to contain and redirect vehicles under nominal impact conditions.

To ensure satisfactory performance for a range of vehicle sizes, rail should be mounted 25-in high as measured from shoulder surface, gutter pan, or widened crown to the center of the rail at the bolt. The rail element shall be spliced midspan between the posts.

Pavement overlays effectively reduce existing rail height. When rail height varies more than 1-in above and 3-in below the 31-in top of rail standard height, steps should be taken to restore the rail to the standard dimension to reduce the possibility of vehicular vaulting or under riding the system. For existing 28-in rail systems, the rail height shall not vary by more than 2-in above and ¼-in below the 28-in top of rail. Existing systems installed with a top rail height less than 27 ¾-in should be upgraded to current standards whenever impacted, repairs needed, or when maintenance budgets permit.

When raising existing metal beam guard fence to the 31-in height, the railing will also need to be adjusted longitudinally and an additional post or 9-ft 4 ½-in rail length will be needed to obtain the mid-span splicing location. Existing bridge transitions may need to be upgraded to current standards or adjusted with a new transition section to obtain the 31-in height. The end treatments may require new materials to adhere to the manufacturer's specifications, such as the breakaway hole and anglestrut locations.

Blockouts

The guard fence is blocked out from the posts with routed timber or composite blockouts (6-in x 8-in). These blockouts minimize vehicle snagging on the posts and reduce the likelihood of a vehicle vaulting over the barrier by maintaining the rail height during the initial stages of post deflection.

It is acceptable to use double blockouts (up to 16-in) to increase the post offset to avoid obstacles such as curbs. There is no limit to the number of posts that can have double blockouts installed, except terminals, unless approved by the manufacturer. Under special circumstances, such as avoiding buried obstacles that are not relocated, it is also acceptable to install triple blockouts (up to 24-in) for one post for every 75-ft of guard fence.

Section 4 — Placement of Guard Fence

Overview

The placement of guard fence pertains to the lateral and longitudinal position.

Lateral Placement at Shoulder Edge or Curb Face

Typically the face of rail is placed at the shoulder edge or curb face throughout most of its length as shown in Figure A-4.

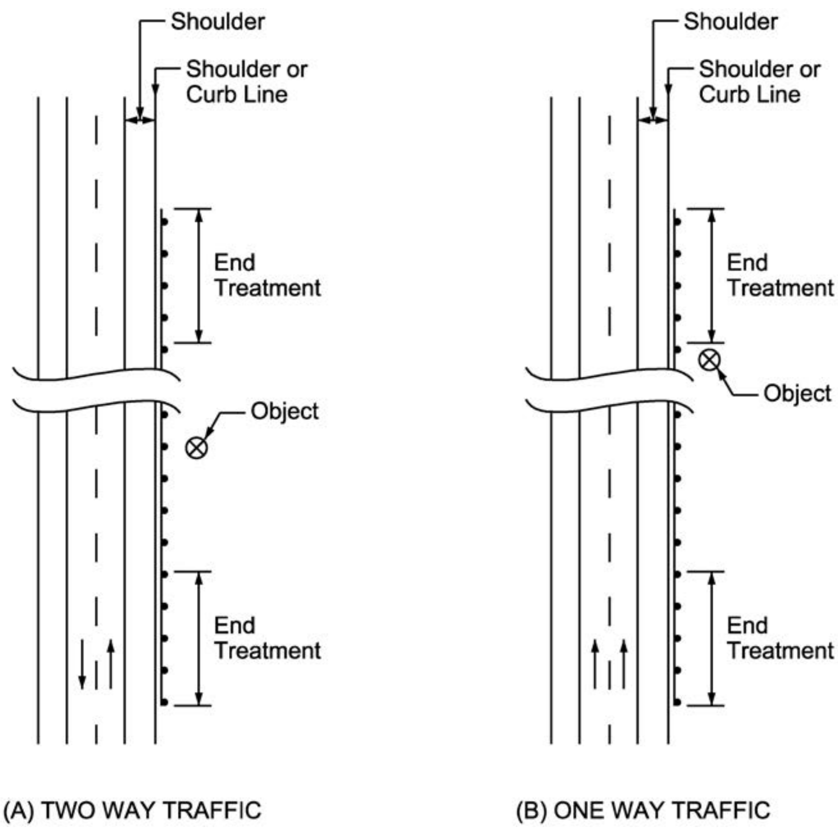


Figure A-4. Placement at Shoulder Edge or Curb Face

Guard fence placed in the vicinity of curbs should be blocked out so that the face of curb is located directly below or behind the face of rail. Rail placed over curbs should be installed so that the post bolt is located 25-in above the gutter pan or roadway surface.

Lateral Placement Away from the Shoulder Edge

In certain instances, it is desirable to place guard fence closer to the obstacle rather than at the shoulder edge or curb face as shown in Figure A-5. Placement in this manner can substantially reduce the length of rail required to shield a given obstacle and minimize the probability of impact, but undesirably, encroachment angles may increase. This manner of placement is most applicable to small areas of concern such as point type obstacles, overhead sign bridge supports, bridge piers, etc.

Care should be exercised in selecting placement location of guard fence with respect to slope conditions to reduce the risk of vaulting or impacting at an undesirable position by errant vehicles. Guard fence may be placed at any lateral location on a side slope only if the slope rate between the edge of the pavement and the face of the barrier is 1V:10H or flatter.

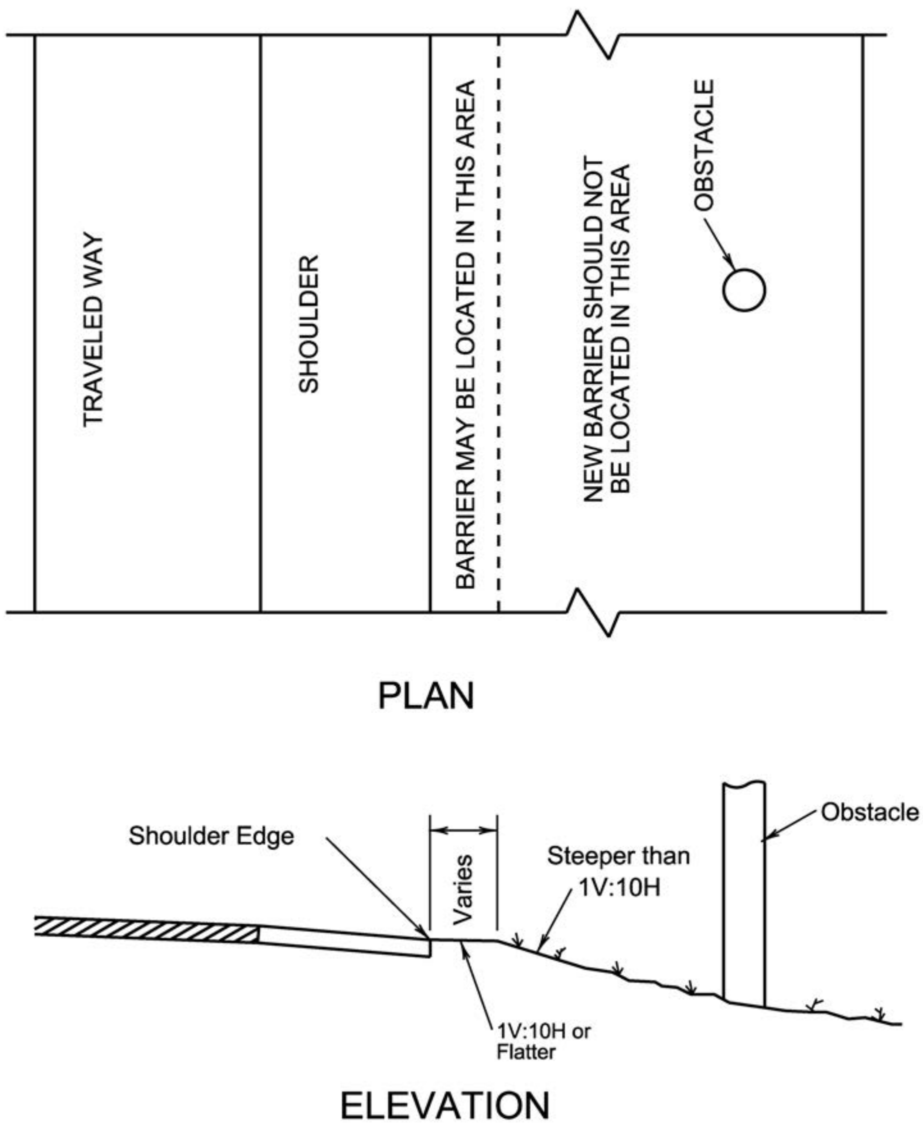


Figure A-5. Location of Roadside Guard Fence.

Deflection Considerations

Guard fence is a semi-rigid barrier system. The amount of dynamic deflection varies primarily with weight of impacting vehicle, its speed, and its encroachment angle. Guard fence should be laterally positioned to provide a clear shoulder width while maintaining a distance from a fixed object that is greater than the dynamic deflection of the rail. Based on crash test data, this barrier-to-object distance should be 4-ft minimum from the back of the post to fixed object or more as diagrammed in Figure A-6. A barrier-to-obstacle distance of 5-ft or more is desirable where conditions permit. In certain circumstances in tightly constrained locations the deflection of the guard fence system can be reduced by reducing the post spacing to either 1/2 or 1/4 post spacing, and applying an appropriate transition to the full post spacing as needed; see *Roadside Safety Pooled Fund Test Report No. 610211-01* (October 2021) for specific guidance.

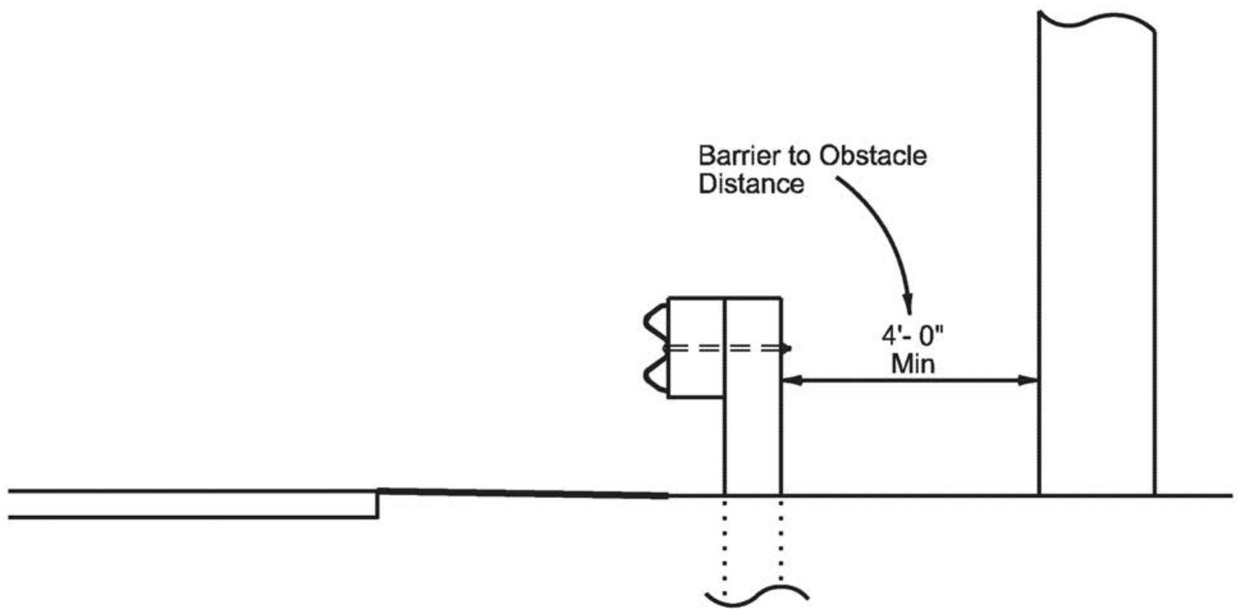


Figure A-6. Allowance for Deflection of Guard Fence

Section 5 — End Treatment of Guard Fence

Overview

Guard fence systems must be anchored at both ends to acceptable end treatments, buried terminals, wingwalls, concrete traffic barriers, etc., so that full tensile strength of the rail may be developed.

Approved end treatments have been developed and are recommended for the upstream end of a guard fence system. These approved end treatments shall be used unless the guard fence terminal is located on the downstream end with respect to adjacent traffic of the guard fence and outside the clear zone for opposing traffic (see Figure A-4). In that case a Downstream Anchor Terminal (DAT) section without offset is acceptable for use.

Section 6 — Determining Length of Need of Barrier

Overview

The shape of the obstacle, its location with respect to travel lanes, the volume of traffic and its corresponding clear zone width are the primary variables influencing length of barrier need. Barrier can be rigid such as a concrete barrier, or semi-rigid, such as metal beam guard fence.

Variables

After all practical means to free the roadside of obstacles have been exhausted, certain areas may remain which constitute an obstacle to errant vehicles. These areas, as illustrated in Figure A-7, will be referred to as an "area of concern."

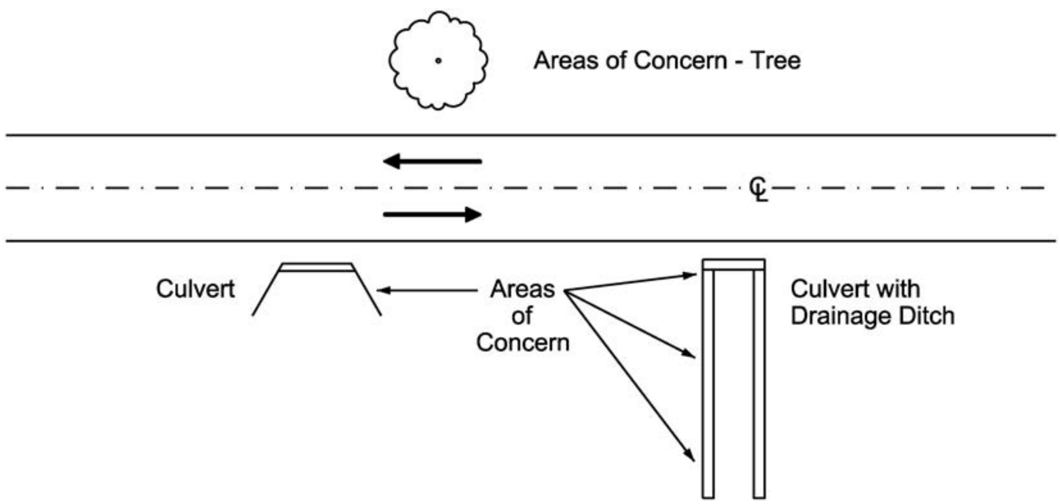


Figure A-7. Areas of Concern

Figure A-8 illustrates the variables of interest in the layout of approach barrier to shield an area of concern. The total length of need is equal to the sum of the following variables:

$$L_{total} = L_u + L_p + L_d$$

Equation A-1.

Where:

L_{total} = Length of guard fence needed, ft

L_u = Guard fence Length Upstream of area of concern, ft

L_p = Guard fence Length Parallel to area of concern, ft

L_d = Guard fence Length Downstream from area of concern, ft

When discussing length of need as it pertains to metal beam guard fence, L_u is the length of guard fence needed to shield the area of concern for adjacent traffic. Upstream refers to the guard fence upstream of traffic adjacent to proposed guard fence. L_d is the length of guard fence needed to protect the opposing traffic. For roadways serving one-way traffic operations, $L_d = 0$. L_d is greater than zero for two-way operations when the area of concern lies within the clear zone of opposing (northbound in Figure A-8) traffic as measured from the centerline pavement markings.

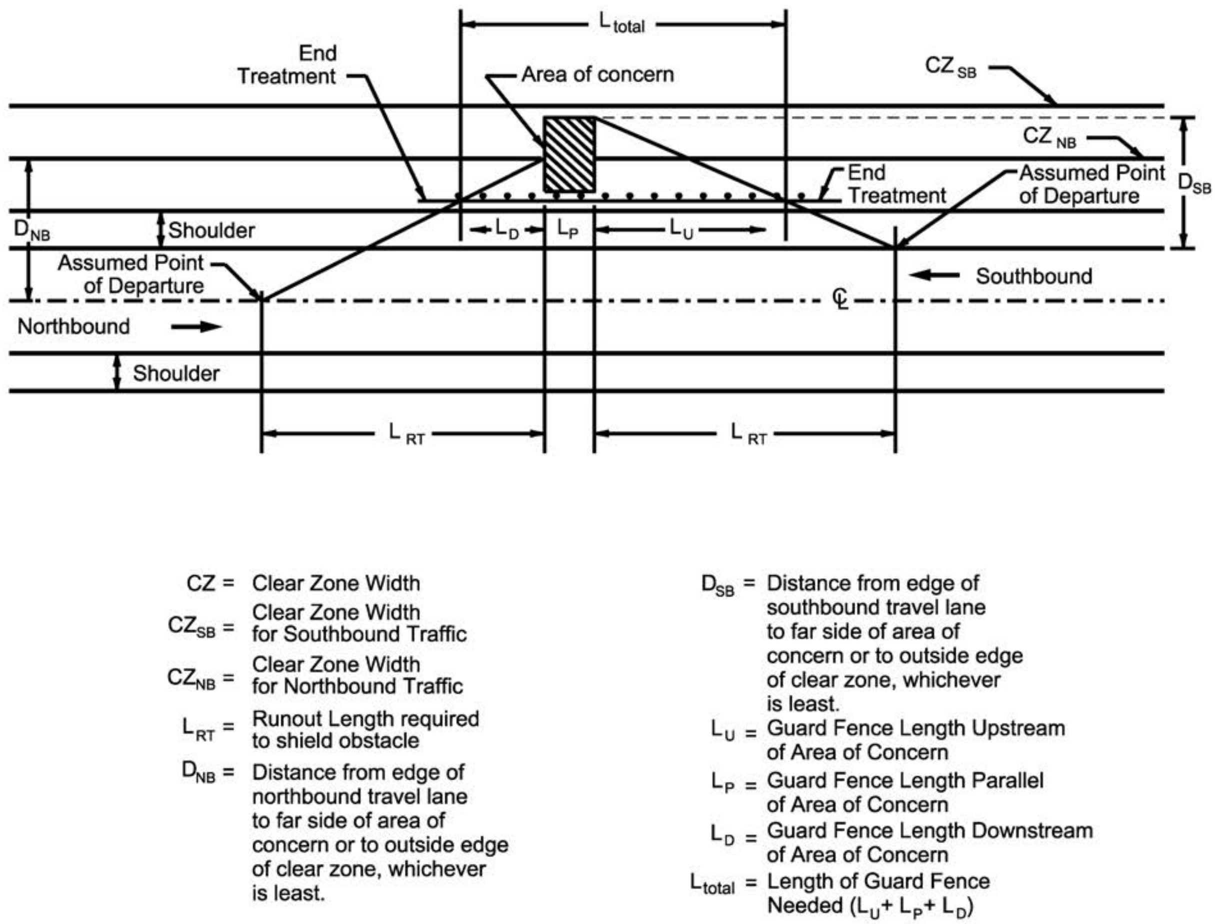


Figure A-8. Variables Involved in Barrier Layout.

In certain instances, judgment should be exercised to supplement design chart solutions and provide for additional safety. For example, high severity fixed objects (e.g., bridge columns) may justify minimum guard fence treatment where located outside the clear zone if geometric conditions (i.e., steep fill slope, outside of horizontal curvature, etc.) increase the likelihood of roadside encroachments. Also, bridge class culverts require protection inside and outside the clear zone. If a bridge class culvert is outside the clear zone, consider increasing the offset of the metal beam guard fence to decrease the length of need. Maintain a 4-ft 0-in minimum distance away from the obstacle and provide a maximum slope of 1V:10H or flatter for the placement of the metal beam guard fence. If the bridge class culvert is outside the clear zone, D_u equals the clear zone distance. If the bridge class culvert is inside the clear zone distance, D_u equals the distance to the outside edge of the bridge class culvert.

Design Equations

To determine the needed length of guard fence or barrier required for a given obstacle, design equations have been formulated for low volume (ADT 750 or less) and higher volume (ADT more than 750) conditions. A clear zone width of 16-ft and length of roadside travel of 200-ft are incorporated in the low volume design equation (for use on roadways when the present ADT volume is 750 or less). Also, if the clear zone required is less than 16-ft and the present ADT is 750 or less, use Equation A-1 for calculating the guard fence length of need.

Table A-2: Equations for Upstream and Downstream Length of Need

ADT ≤ 750	$L_u = 200 - \frac{200}{D_u} x G_u$ *Equation A-2.*
	$L_d = 200 - \frac{200}{D_d} x G_d$ *Equation A-3.*
ADT > 750	$L_u = 250 - \frac{250}{D_u} x G_u$ *Equation A-4.*
	$L_d = 250 - \frac{250}{D_d} x G_d$ *Equation A-5.*

Where:

L_u = Length of guard fence needed (upstream of area of concern), ft

L_d = Length of guard fence needed (downstream of area of concern), ft

D_u = Distance from edge of travel lane to far side of area of concern or to outside edge of clear zone, whichever is least, ft (for upstream direction of traffic)

D_d = Distance from edge of travel lane to far side of area of concern or to outside edge of clear zone, whichever is least, ft (for opposing direction of traffic)

G_u = Guard fence offset from edge of travel lane adjacent to proposed guard fence, ft

G_d = Guard fence offset from edge of opposing direction of travel lane (centerline), ft

For low volume conditions, if the clear zone width of 16-ft is met or exceeded, L=0.

For higher volumes, a clear zone width of 30-ft and length of roadside travel of 250-ft are incorporated into the design equation (for use on roadways when the present ADT volume is more than 750 or the recommended clear zone is greater than 16-ft).

For high volume conditions, if the clear zone width (30-ft) is met or exceeded, L=0.

Using Design Equations to Determine Length of Guard Fence

Before determining length of guard fence, the designer should assemble the following pertinent data:

- Present ADT volume;
- Clear zone (horizontal clearance);
- Traffic operations (one-way or two-way);
- Lateral and longitudinal dimension of the area of concern;
- Shoulder width;
- Offset distance of the area of concern from the edge of travel lane (including from the centerline markings for two-way traffic operations);
- Design slope conditions, (i.e. will slopes be 1V:10H or flatter);
- Placement location (alongside shoulder vs. near object, flared, etc.); and
- Presence of other nearby areas of concern which should be considered simultaneously.

Once this design data has been assembled, the equation for length of guard fence can be used.

Where the prescribed length of the guard fence cannot be installed at a bridge end due to an intervening access point such as an intersecting roadway or driveway, the length of guard fence may be interrupted or reduced. This change in length is acceptable only in locations where the Department must meet the obligation to provide access and this access cannot be reasonably relocated. Alternative treatments in these situations include installing an appropriate radius rail, terminating the guard fence prior to the access location with an appropriate end treatment and continuing the guard fence beyond the access location if necessary, or using an alternate bridge end treatment. The selected treatment should consider potential sight line obstructions, crash history at the site, cost, and maintenance associated with the selected treatment. Reduced guard fence length to accommodate access points will not require a design exception or a design waiver.

Section 7, Example Problems provides example problems and solutions using the design equations. The guard fence lengths produced by the equations should be rounded up to an even length of guard fence. In circumstances where site conditions permit, the rounded-up length of need should terminate at the end of guard fence; any additional length of need component available from an end attenuator should be considered an additional buffer.

Section 7 — Example Problems

Example Problem 1

Given: A rural two-lane collector highway containing 6-ft wide shoulders and a current ADT of 500 is illustrated in Figure A-9. The area of concern is a 16-ft design clear zone that includes 1V:2H side slopes on a 10-ft high embankment section that is 125-ft in length alongside the highway.

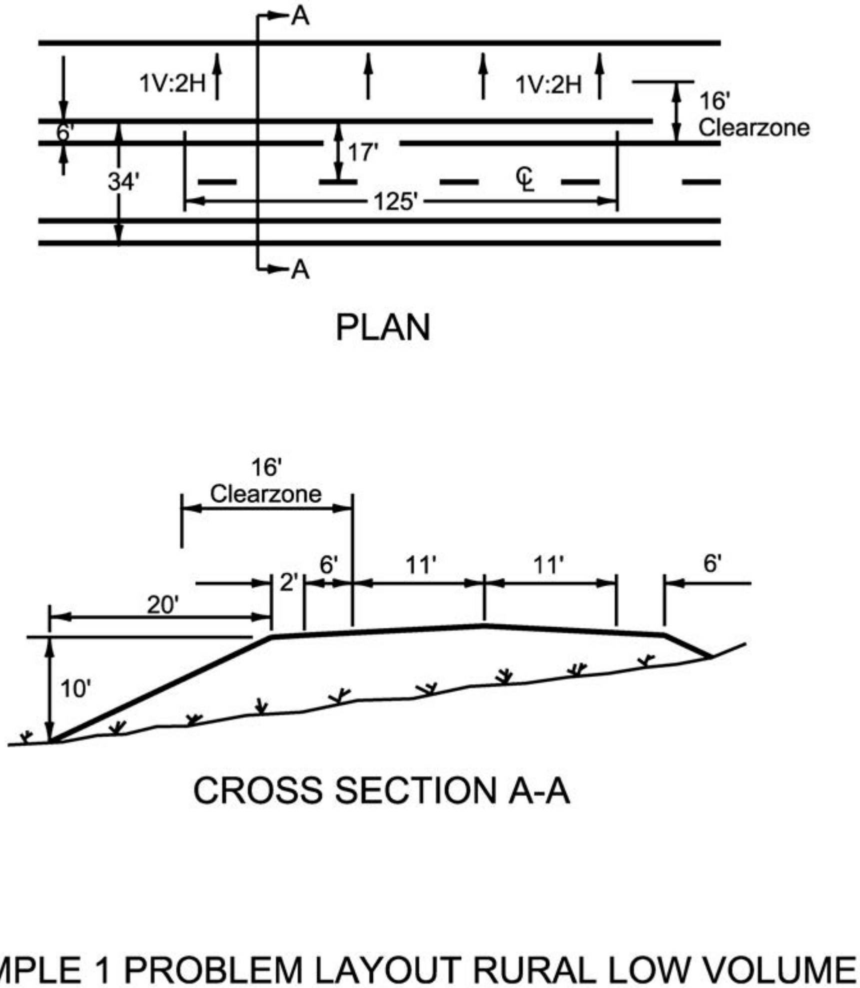


Figure A-9. Example 1 Problem Layout Rural Low Volume.

Solution: From the information above and referring to Figure A-1 it is determined that a "rail is needed." As shown in Equation A-1, the length of need is $L_{total} = L_u+L_p+L_d$. From the given information, L_p = 125-ft. Because the ADT is less than 750, Equations A-2 and A-3 are used to solve for L_u and L_d, respectively (if necessary).

For the upstream direction, the area of concern is the full (16-ft) clear zone width and the guard fence offset (G_u) is 6-ft. Substituting in Equation A-2.

$$L_u = 200 - \frac{200}{16} \times 6 = 125\text{ft}$$

A placement of guard fence alongside the 6-ft wide shoulder results in L_u = 125-ft.

Referring to Figure A-9, the length of guard fence needed in the downstream is zero because the offset distance from the edge of the travel lane (centerline marking) to the area of concern is greater than the design clear zone (17-ft greater than 16-ft). Therefore, L_d is zero.

The design placement is shown in Figure A-10 including 125-ft of guard fence adjacent to the obstacle plus 125-ft shielding traffic adjacent to proposed guard fence upstream of the obstacle. These lengths of need do not include end treatments.

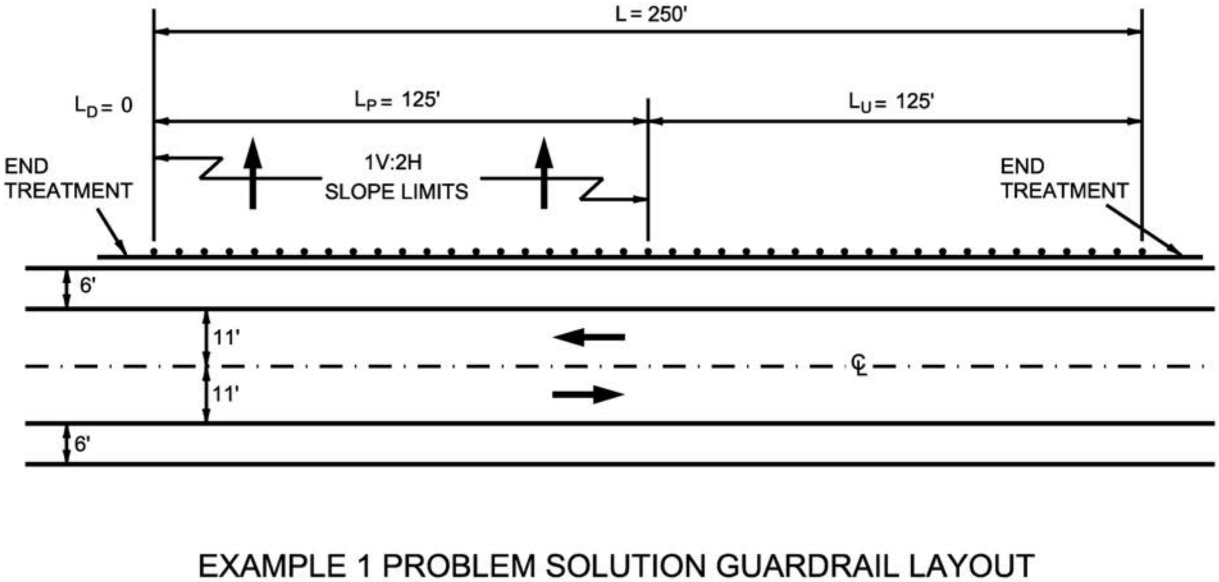


Figure A-10. Example 1 Problem Solution Guard Fence Layout

Example Problem 2

Given: A rural two-lane arterial highway containing a shoulder width of 8-ft and a current ADT of 3500 is illustrated in Figure A-11. The areas of concern are bridge bents located 5-ft from the edge of shoulder. The side slopes are 1V:6H.

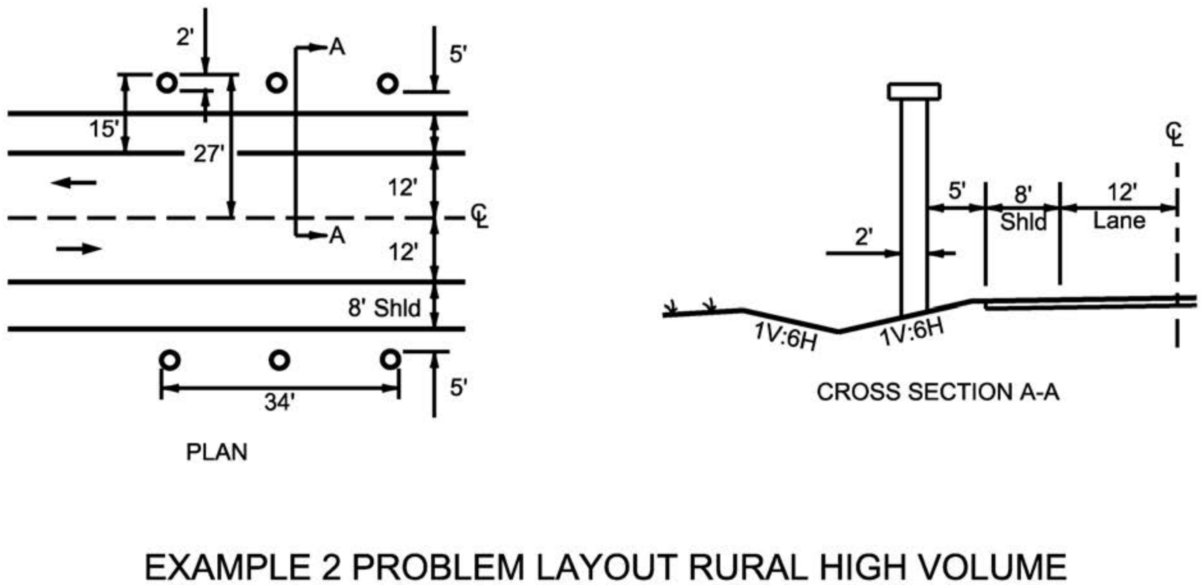


Figure A-11. Example 2 Problem Layout Rural High Volume.

Solution: Referring to Table A-1: General Applications of Conditions for Roadside Barriers bridge piers within the clear zone (30-ft in this case) indicates guard fence placement for the north side of the roadway displayed in Figure A-11. As shown in Equation A-1 the length of need is $L_{total} = L_u+L_p+L_d$. Therefore, L_p is 34-ft from the given (see Figure A-11) information. Because the ADT is greater than 750, Equations A-4 and A-5 are used to find L_u and L_d (if necessary), respectively:

$$L_u = 250 - \frac{250}{15} \times 8 = 116.5\text{ft}$$

Substituting in the equation, the upstream length (L_u) is 116.5-ft if placement is at the shoulder edge.

The downstream (westbound traffic) length of guard fence is also determined by substituting into Equation A-5:

$$L_d = 250 - \frac{250}{27} \times 20 = 65\text{ft}$$

L_d is 65-ft as shown above, based on the shoulder edge placement. For westbound traffic, the centerline is the edge of the travel lane and thus guard fence offset (G) is 20-ft (12-ft lane plus 8-ft shoulder) from the edge of the travel lane.

Total length of guard fence, $L_u+L_p+L_d$, thus is 116.5-ft + 34-ft + 65-ft or 215.5-ft; or, rounded to an even length of guard fence, 225-ft.

The solution for the south side of the roadway yields the same results; hence placement should be as shown in Figure A-12.

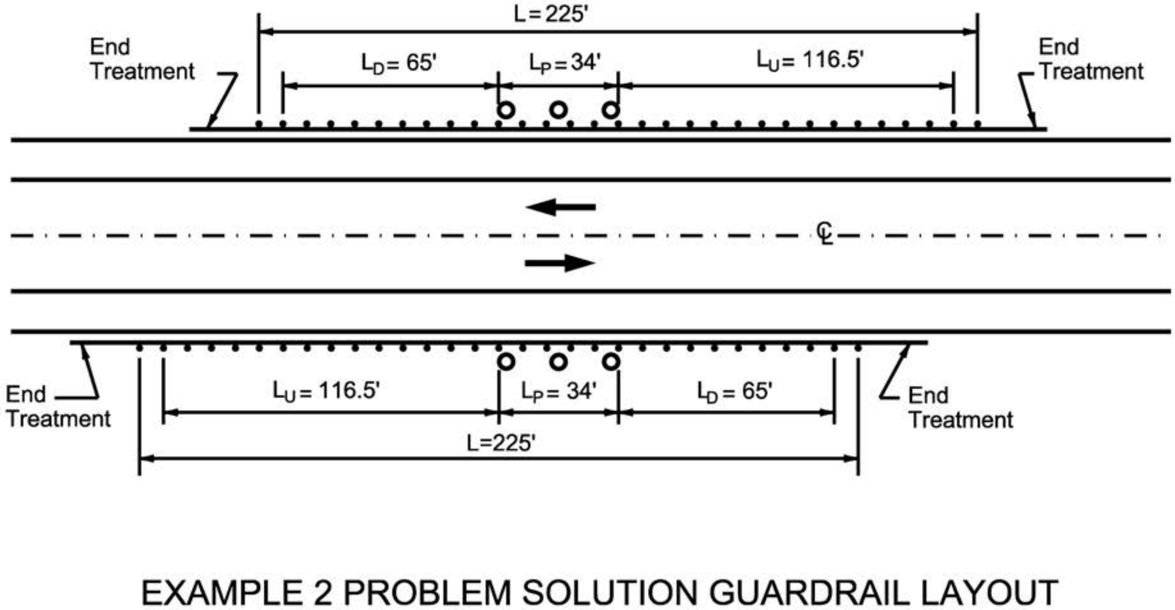


Figure A-12. Example 2 Problem Solution Guard Fence Layout.

Example Problem 3

Given: A divided (76-ft median) highway with 4-ft left and 10-ft right shoulder widths is illustrated in Figure A-13. The median slopes are 1V:10H, and the outside side slopes are 1V:6H. The cross-sectional design allows for the addition of a future lane on the median side of the present lanes. The areas of concern are overhead sign bridge supports offset 25-ft left and 18-ft right from edge of the travel lanes as shown below. The ADT is 10,000.

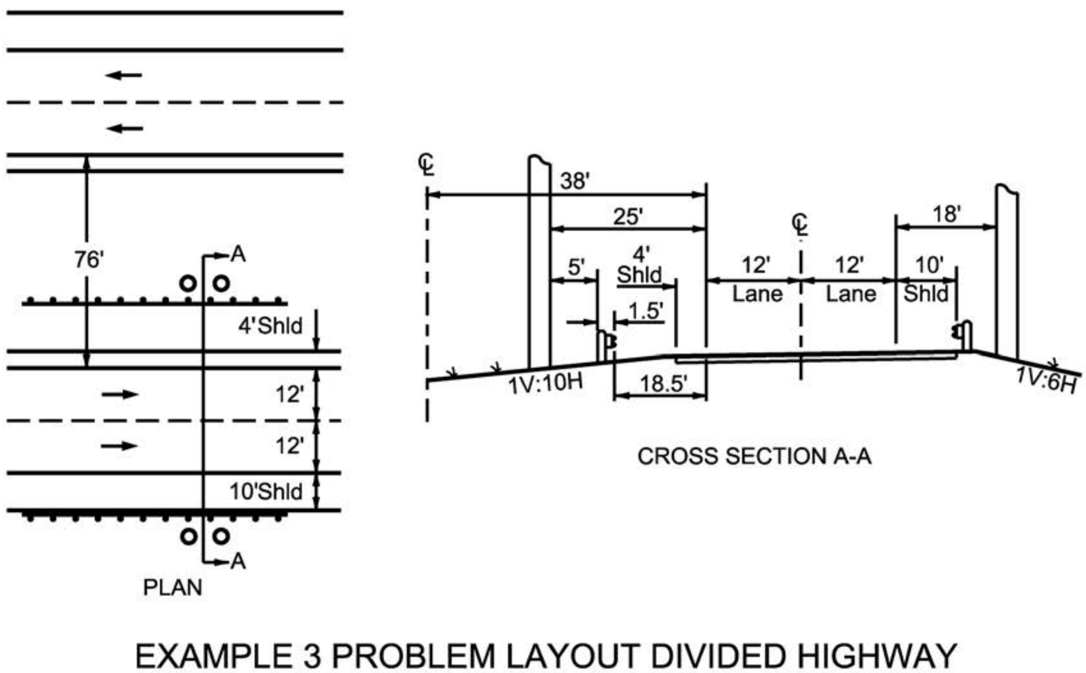


Figure A-13. Example 3 Problem Layout Divided Highway.

Solution: Crash cushions in lieu of guard fence should be considered, particularly for facilities with higher than 10,000 ADT. For this example problem assume crash cushions are not cost effective.

Because the median is sloped at 1V:10H, as shown in Figure A-13, guard fence may be placed thereon (see Figure A-5). Therefore, place the guard fence such that the back of the posts are 5-ft in front of the median overhead sign bridge support to allow for deflection, i.e., 20-ft from the edge of the travel lanes (including the 1.5-ft from the back of the post to the face of the rail).

Referring to Equation A-1, $L_{total} = L_u + L_p + L_d$. For one-way traffic operations, $L_d = 0$; furthermore, for the overhead sign bridge support $L_p = 0$. Equation A-4 is used to find L_u because ADT is greater than 750:

$$L_u = 250 - \frac{250}{25} \times 18.5 = 65\text{ft}$$

Equation A-6.

For the median side, L_u = 65-ft (rounded to 75-ft to conform to even lengths of guard fence) based on parallel placement for the full length of need, and placement on the 1V:10H slope 5-ft in front of the fixed object. In contrast, parallel placement at the shoulder edge would have required over 200-ft of guard fence.

For the right side of traffic, guard fence must be placed at the shoulder edge (Reference Figure A-5). Substituting in Equation A-2 to determine L_u:

$$L_u = 250 - \frac{250}{18} \times 10 = 111\text{ft}$$

Equation A-7.

Using parallel placement for the entire length, L_u = 111-ft (which should be rounded to 125-ft to conform to even lengths of guard fence).

Using parallel placement for the entire length of guard fence for both the median and left side, placement is as shown in Figure A-14.

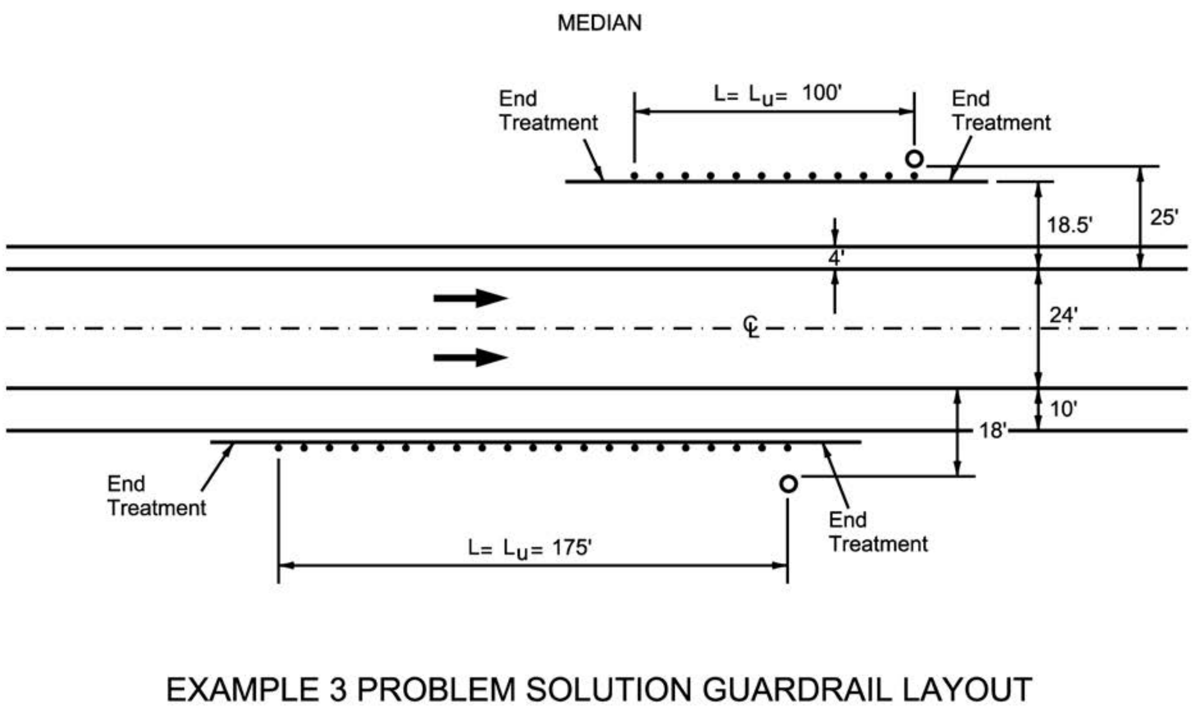


Figure A-14. Example 3 Problem Solution Guard Fence Layout.

Section 8 — Median Barrier

Overview

Median barriers are used to reduce median crossover vehicle encroachments and to protect against continuous longitudinal obstacles, and can generally be categorized as:

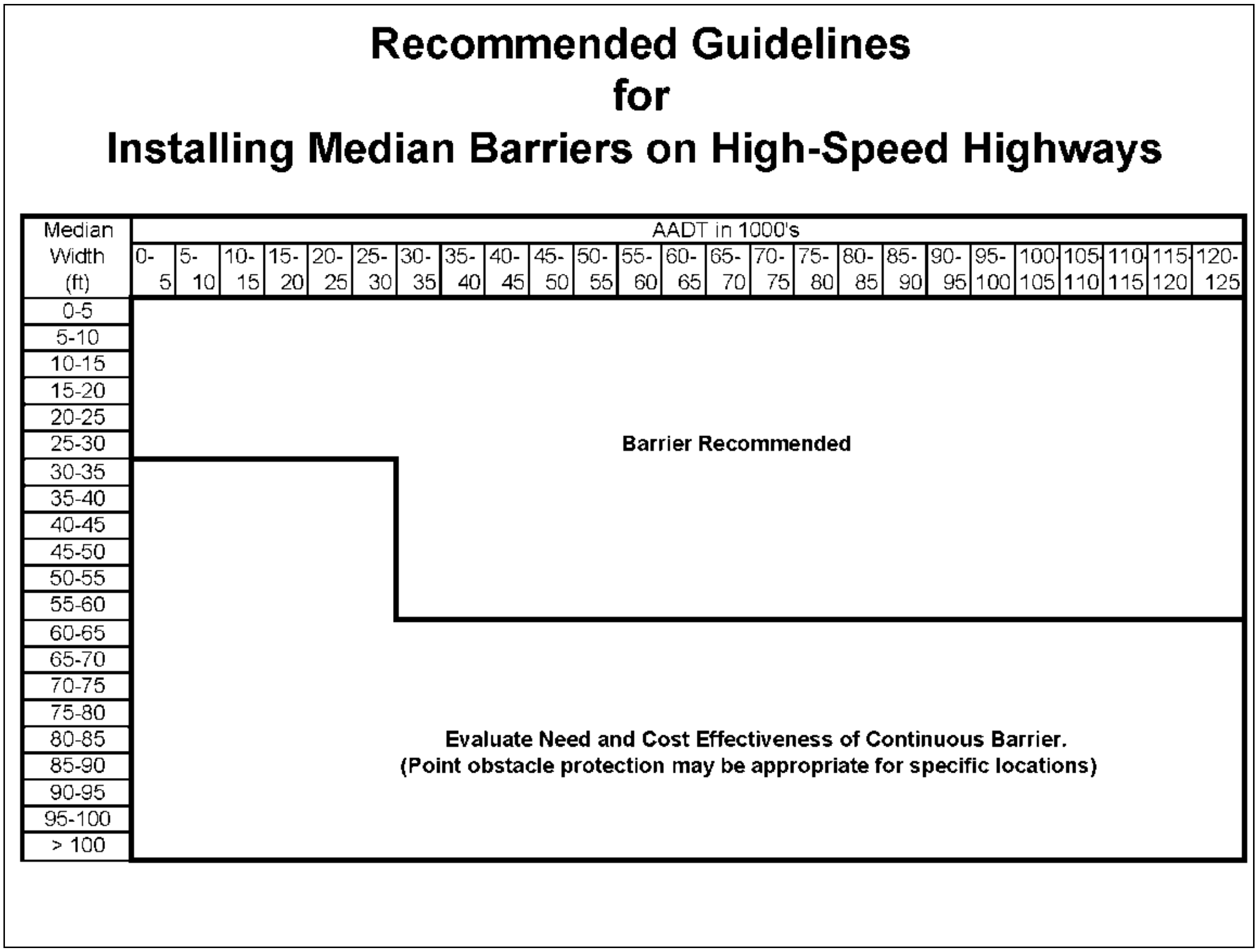
- Concrete barriers (such as F-shape or single sloped); or
- High-tension cable barrier systems.

The utilization of other median barriers, such as metal beam guard fence, may be appropriate based on the need to protect point obstacles in the median, such as overhead sign supports, etc. (See Sections 1-7).

Application

On high-speed highways, median barriers should be considered based on the criteria shown in Table A-3. Flush medians or frequent crossovers may preclude the use of median barriers based on an engineering analysis of individual locations.

Table A-3: Recommended Guidelines for Installing Median Barriers on High-Speed Highways



The criterion is divided into two different zones by various combinations of average annual daily traffic and median width.

- **Barrier Recommended:** Barrier should be installed.
- **Evaluate Need and Cost Effectiveness of Continuous Barriers. (Point obstacle protection may be appropriate for specific locations):** An engineering analysis should be performed to determine if barrier is needed for reducing the occurrence of cross-median encroachments (crashes). This analysis may consider the following:
  - Type of median (flush, depressed [V-ditch or flat-bottom]);
  - Width of the median (measured from edges of travel lane for opposing directions of travel);
  - Traffic volumes, including estimated traffic growth and percent trucks;
  - Types and severity of crashes;
  - Posted speed limit;

- Type of facility, including controlled access or non-controlled access with crossovers;
- Roadway alignment;
- Ramp locations; and
- Elimination of barrier gaps.

Continuous barrier should be limited to areas where it is needed to reduce cross-median incidents and should not be used for point obstacles (i.e. overhead sign bridges, etc.), or in areas of lesser concern (i.e. wider medians, forested areas, etc.).

Placement

As a general rule, a barrier should be placed as far from the traveled way as possible while maintaining the proper operation and performance of the system. The more lateral offset afforded a driver, the better the opportunity for the driver to regain control of the vehicle in a traversable median and avoid a barrier impact. The placement of concrete barrier adjacent to narrow shoulders is discouraged. It is recommended that a minimum clear distance of 12-ft be maintained from the edge of the travel lane and a cable barrier to prevent incidental contact with the barrier being too close to the travel lane. Also, due to the deflection of the cable barrier, it is recommended that a minimum distance of 12-ft be maintained between the cable barrier and any obstruction being protected.

Slopes

Where possible, barriers should be installed on relatively flat, unobstructed terrain (1V:10H or flatter). Barriers may also be placed on 1V:6H maximum slopes as shown in Figure A-15. The centerline axis of the barrier shall be vertical.

From the perspective of barrier performance alone, it is acceptable placement practice to locate the barrier at, or within 1-ft, of the bottom of the ditch line. If it is desirable to offset the median barrier more than one foot from the bottom of the ditch line to avoid drainage issues (potential for erosion, etc.), the barrier can be placed anywhere along the 1V:6H median slope, provided it is located at least 8-ft from the bottom of the median ditch line. This offset from the bottom of the ditch line reduces the potential for the vehicle striking the barrier too low for the barrier to function properly.

If the slopes in the median are steeper than 1V:6H and barrier is needed, consideration should be given to regrading the slopes to meet the requirements or to filling in the median to place a split-level concrete barrier.

If re-grading or other options are not feasible, placement of cable barrier on slopes up to 1V:4H is an alternative to consider. Contact the Design Division for assistance in locating the cable barrier at the proper location along slopes steeper than 1V:6H. While not desirable, some median configurations may require barrier placement on both sides of the median to provide the proper protection.

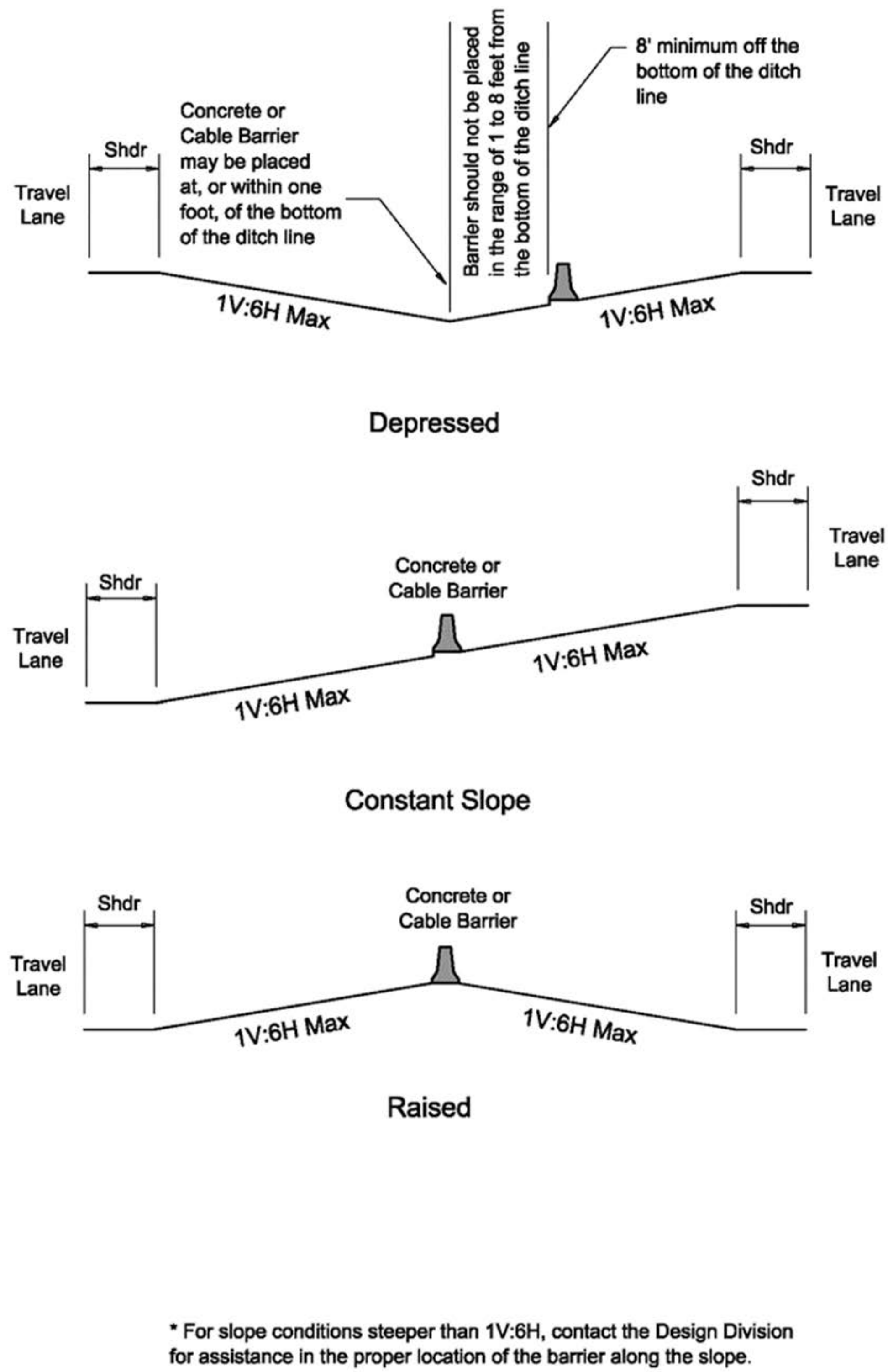


Figure A-15. Desirable Barrier Placement in Non-Level Medians.

Additional Cable Median Barrier Guidance

The cable barrier is for median use only and on medians greater than 25-ft. Median widths of 25-ft or less require the use of a more rigid barrier such as concrete median barrier.

The selection of Test Level 4 (TL-4) cable barrier over Test Level 3 (TL-3) cable barrier is at the district's option. FHWA policy requires that all roadside appurtenances such as traffic barrier and barrier terminals used on the National Highway System (NHS) meet the performance criteria contained in the National Cooperative Highway Research Program (NCHRP) Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features or the updated testing procedures found in The Manual for Assessment of Safety Hardware (MASH). Safety features approved to a minimum TL-3 are acceptable for highspeed arterial highway. Any decision to use TL-4 tested barrier should be made based on-site conditions and traffic mix, using engineering judgement.

The vertical alignment of the system is essential since the location of the bottom cable with respect to the ground is critical to capturing smaller vehicles. Special attention should be placed on sag vertical alignments. The cables and/or posts placed in sockets are free standing (not held down by the system) and will come to a taut elevation between two tangent points when the cable is tensioned creating a larger distance from the ground line to the bottom cable than allowed by the manufacturer's installation manual. Sag vertical alignments with radii of less than a K-Value of 11 should be avoided.

The placement of the system should also take into consideration the drainage facilities located in the median. Cross drainage structures with less than 36-in of cover pose a challenge for placing posts. Structures of less than 16-ft can be spanned and construction of these runs of cable should take these structures into account prior to setting post locations.

If an obstruction is currently protected by MBGF and there will be minimum of 12-ft clearance from the proposed cable barrier to the obstruction, the MBGF may be removed. If there is less than 12-ft clearance from the proposed cable barrier to the obstruction, it is recommended that the MBGF be left in place, and the cable barrier be placed such that there is a minimum of 4-ft (5-ft preferred) from the back of the MBGF posts to the barrier. This allows for deflection of the MBGF without engaging the cable barrier. Cable barrier should be a minimum 5-ft behind SGT's to allow for extrusion and gating of the end treatment.

Cable barrier systems deflection is based on it being installed along a tangent or when struck on the "concave" side (from the inside of a curve). When it is struck in the "convex" side (from the outside of the curve) the barrier must deflect enough to redevelop a concave condition. Placement of the barrier on the convex side is also recommended to allow maximum median availability for deflection (see Figure A-16). In order to minimize the length over which this occurs, closer post spacing through these curves is recommended (see Table A-4).

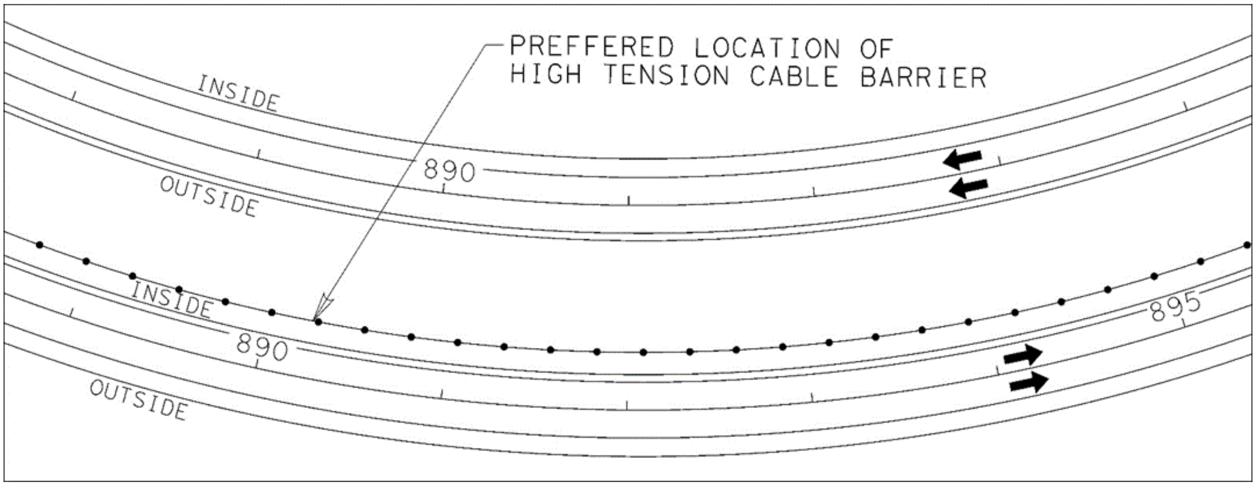


Figure A-16. Desirable Cable Barrier Placement in a Curve.

Table A-4: Recommended Cable Barrier Radius and Post Spacing in a Curve

Radius (ft)	Post Spacing
650-2,500	6-ft, 8-in
2,501-5,500	10-ft
> 5,500	Standard Recommendation

A recommended maximum run of cable barrier between anchors should be approximately 10,000-ft. This length allows for proper tensioning of the system and reasonable construction installation time to get a run-in operation. Runs of shorter and longer lengths between anchors may be appropriate in specific locations and each run should be determined to meet the field situations.

Section 9 — Emergency Crossovers

Overview

Emergency crossovers may be provided when needed to facilitate official vehicles. Coordination with local and state law enforcement and emergency services personnel is recommended to identify roadway sections where emergency crossovers may be necessary.

Location

Refer to Chapter 7 Section 6, Emergency Crossovers for guidance when selecting a location for an emergency crossover.

Construction

When ending a run of cable barrier, the cable barrier terminals should be located, when possible, behind some protection such as the MBGF, leaving adequate distance to allow an emergency vehicle to maneuver around if necessary. See Figure A-17.

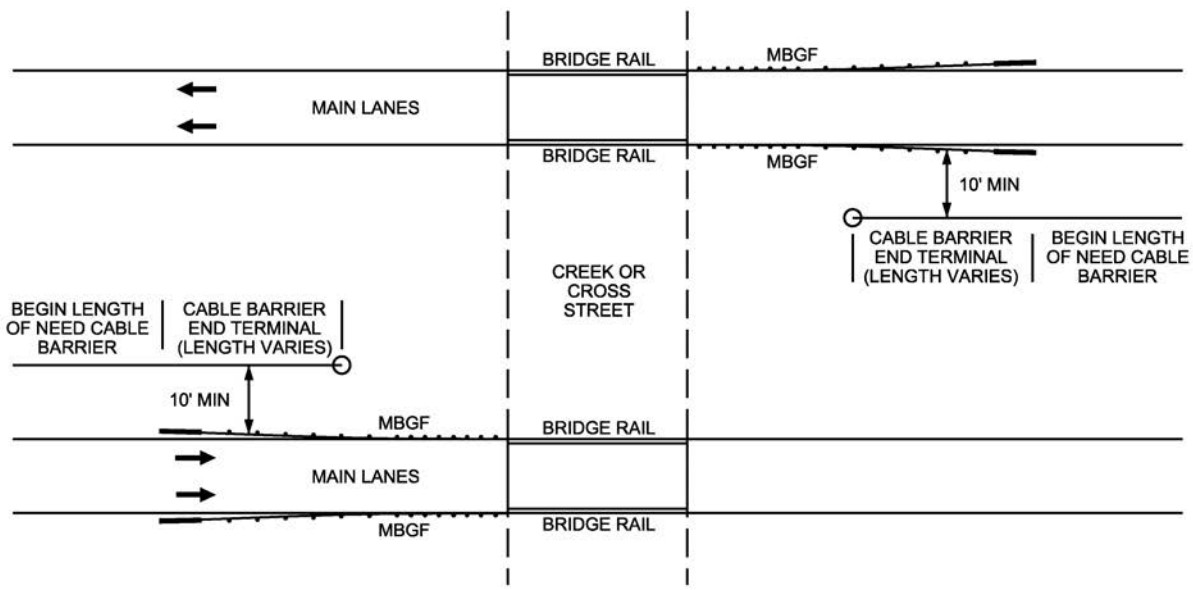


Figure A-17. Cable Terminals Behind Metal Beam Guard Fence

The terminals can be placed in locations with no protection, but since they provide the anchorage for the cable barrier system, protecting them from possible hits is recommended. These terminals are also gating (meaning they will not prevent a vehicle from going through).

When switching the cable barrier from one median side to the other and the terminals are not protected, overlapping the runs of cable barrier is recommended to provide adequate protection from possible crossovers if the median is wide enough to allow emergency vehicles to utilize it as an effective emergency crossover. (See Figures A-18 through A-19). The cable barrier anchor terminal should be protected behind guard fence or other barriers from potential impacts. An exposed cable barrier anchor terminal will render the cable system ineffective if damaged by an impact.

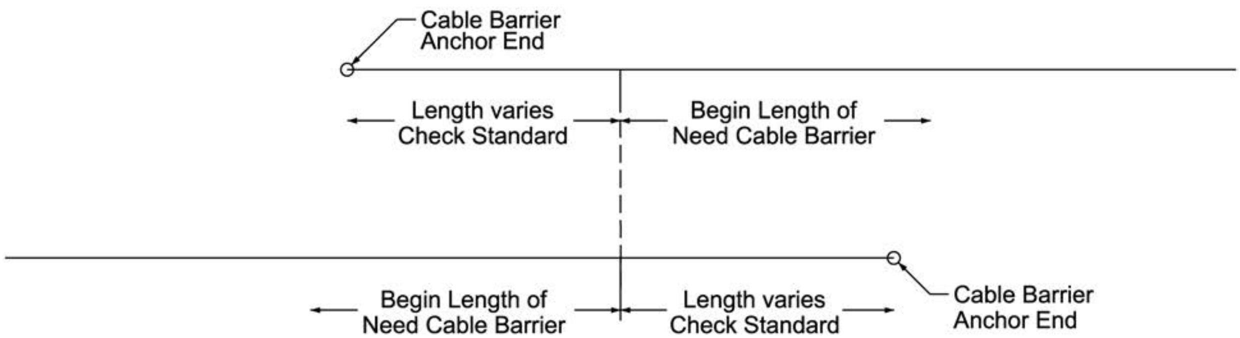


Figure A-18. Recommended Cable Barrier Lap Length

Another typical layout for emergency crossovers may be as shown below.

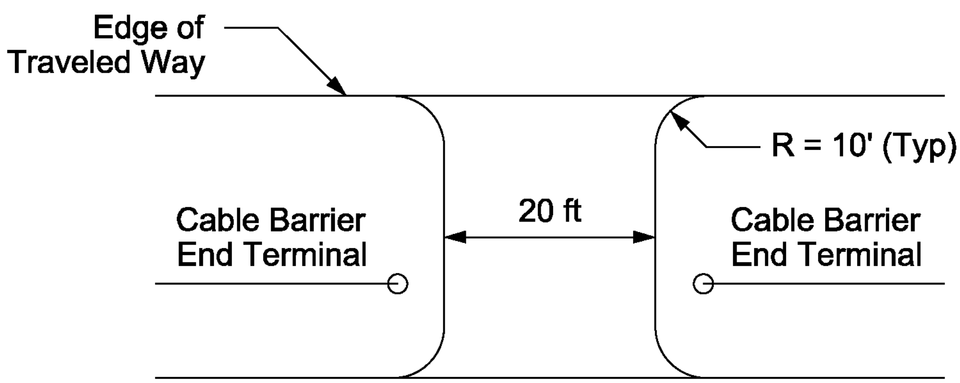


Figure A-19. Another Typical Layout for Emergency Crossovers

- Emergency crossovers should be an all-weather surface. It is recommended that they be constructed with a surface treatment that does not invite use. Grade 1 or 2 aggregate or bladed recycled asphalt pavement (RAP) has provided an adequate low cost surface in some applications.
- Emergency crossovers should be approximately 20-ft with return radii of 10-ft. Wider crossovers invite non-emergency use and should only be constructed after an engineering study of the site.
- To be inconspicuous to main lane traffic, the surface should be depressed below the shoulder level, if possible.

Appendix B — Treatment of Pavement Drop-offs in Work Zones

Contents:

Section 1 — Overview

Scope

These guidelines apply to construction zone work where continuous pavement edges or drop-offs exist parallel and adjacent to a lane used for traffic. These guidelines do not apply to short-term operations. The *Texas Manual on Uniform Traffic Control Devices* (*TMUTCD*) defines short-term operations as daytime work from one to twelve hours.

These guidelines do not constitute a rigid standard or policy; rather, they are guidance to be used in conjunction with engineering judgment.

Types of Treatment

Treatment may consist of either or both of the following:

- Warning devices (such as signs or channelizing devices), and
- Protective barriers (such as concrete traffic barriers or metal beam guard fence).

Factors Affecting Treatment Choice

The type of treatment (warning device or protective barrier or both) selected depends on several factors, including engineering judgement. These guidelines are based on the factors presented in Table B-1.

Table B-1: Factors Considered in the Guidelines

Factor	Definition	Notes
Edge condition	Slope of the drop-off	For more information, see “Edge Condition” subheading below.
Lateral clearance	Distance from the edge of the travel lane to the edge condition	See Figure B-1 for description.
Edge height	Depth of the drop-off	See Figure B-1 for description.

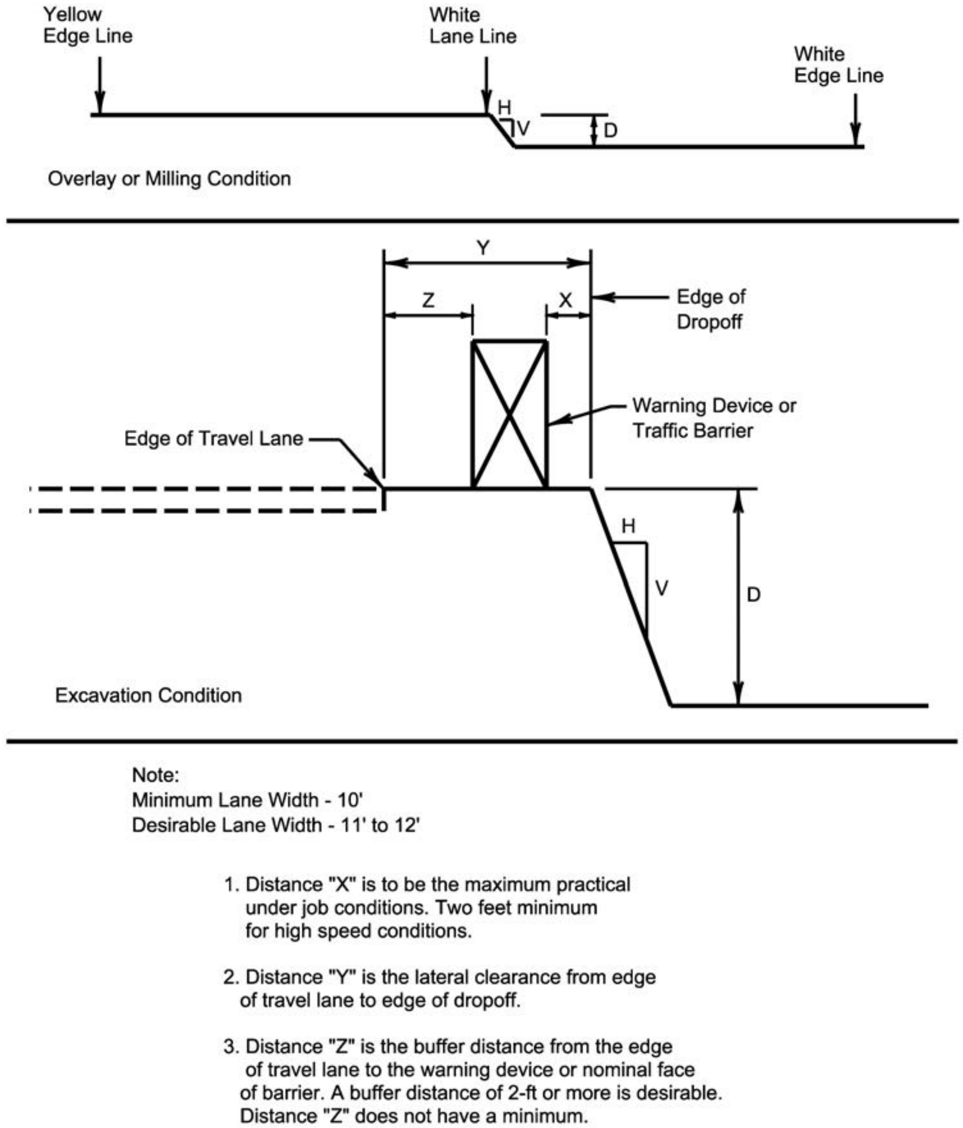


Figure B-1. Definition of Terms.

In addition to the factors considered in the guidelines, each construction zone drop-off situation should be analyzed individually, taking into account other variables, such as:

- Traffic mix;
- Posted speed in the construction zone;
- Horizontal and vertical curvature;
- Practicality of treatment options;
- Driveways and access;
- Safety impacts and crash history; and
- Illumination.

In urban areas where speeds of 30 mph or less can be predicted for traffic in a particular construction zone, there may be a lesser need for signing, delineation, and barriers. Regardless, sharp 90-degree edges greater than 2-in in height, if located within a lateral offset distance of 6-ft or less from a traffic lane, may indicate a higher level of treatment.

If distance *Y* (as described in Figure B-1) is less than 3-ft, use of positive barrier may not be feasible. In such a case, if a positive barrier is required (according to Figure B-2), then consider one of the following:

- Moving the lane of travel laterally to provide the needed space, and
- Providing an edge slope such as Edge Condition I.

Edge Condition

"Edge condition" refers to the slope of the drop-off. Table B-2 describes three edge condition types used in these guidelines. These edge conditions may be present between shoulders and travel lanes, between adjacent or opposing travel lanes, or at intermediate points across the width of the paved surface. Due to the variability in construction operations, tolerances in the dimensions shown in the figures may be allowed by the engineer.

Table B-2: Edge Condition Types

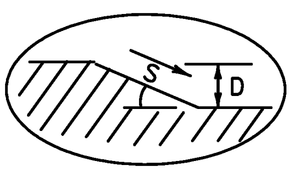
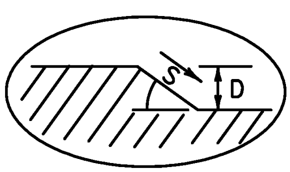
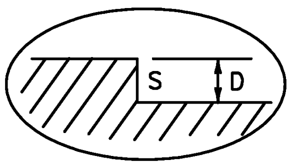
Condition Type & Description	Notes
Edge Condition I  *S* = 3:1 or flatter slope rate (*H:V*)	Most vehicles are able to traverse an edge condition with a slope rate of 3 to 1 (horizontal to vertical) or flatter. The slope must be constructed with a compacted material capable of supporting vehicles.
Edge Condition II  *S* = 2.99:1 to 1:1 slope rate (*H:V*)	Most vehicles are able to traverse an edge condition with a slope between 2.99 to 1 and 1 to 1 (horizontal to vertical) as long as *D* does not exceed 5 inches. Undercarriage drag on most automobiles will occur as *D* exceeds 6 inches. As *D* exceeds 24 inches, the possibility of rollover is greater for most vehicles.

Table B-2: Edge Condition Types

Condition Type & Description	Notes
Edge Condition III  *S* is steeper than 1:1 slope rate (*H:V*)	Slopes steeper than 1 to 1 (horizontal to vertical) where *D* is greater than 2 inches can present a more difficult control factor for some vehicles, if not properly treated. For example, in the zone where *D* is greater than two and up to 24 inches, different types of vehicles may experience different steering control at different edge heights. Automobiles might experience more steering control differential in the greater than 2 and up to 5 inch zone. Trucks, particularly those with high loads, have more steering control differential in the greater than 5 and up to 24 zone. As *D* exceeds 24 inches, the possibilities of rollover are greater for most vehicles. NOTE: Milling or overlay operations that result in Edge Condition III should not be in place without appropriate warning treatments, and these conditions should not be left in place for extended periods of time.

Guidelines for Treatment

The “Treatment for Various Edge Conditions” standard sheet shows the treatments for given combinations of edge condition, lateral clearance, and edge height. Remember to consider other factors listed above and use engineering judgment. The standard sheet requires the Engineer's seal, signature, and date.

Edge drop-offs greater than 2-in immediately adjacent to traffic should not be left overnight.

Careful consideration should be given to allowing 1-in to 2-in vertical longitudinal joints between lanes or directly adjacent to the travel lane. Vertical joints greater than 1-in are particularly challenging for motorcyclist and bicyclists to traverse. If a 2-in vertical longitudinal joint is to be used in or adjacent to traffic, consider using the notched-wedge joint as shown in Figure B-2.

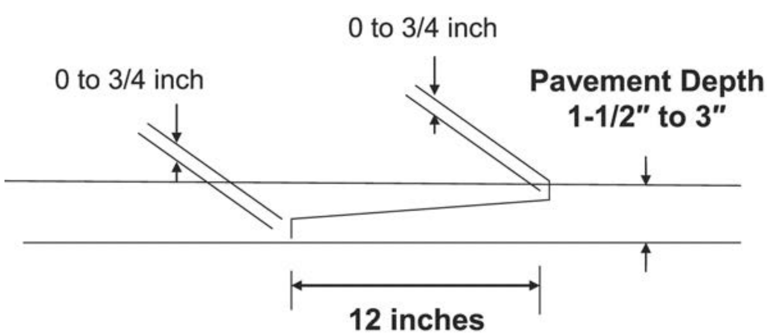
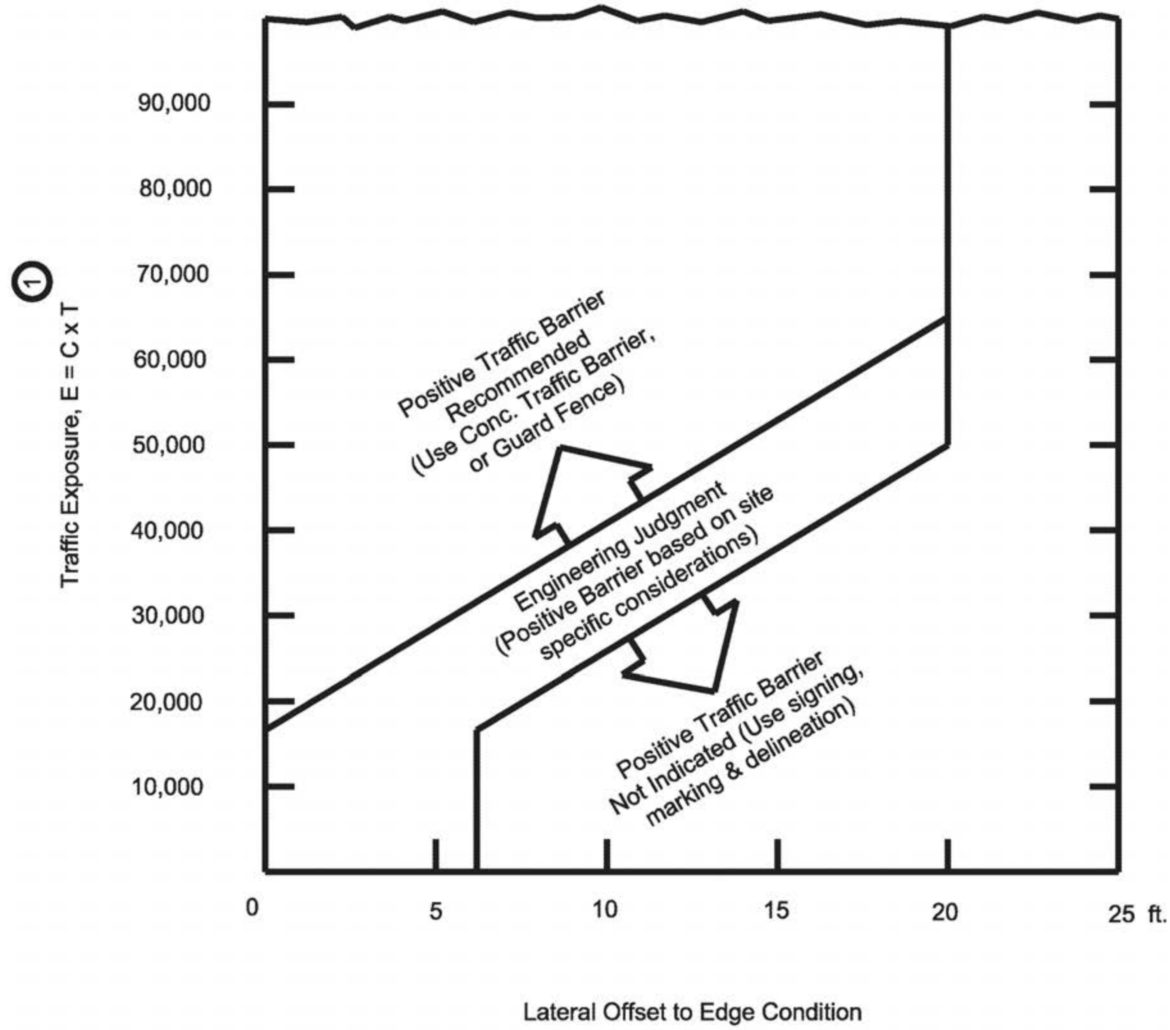


Figure B-2. Notched-Wedge Joint Detail.

Refer to the Construction Divisions technical advisory report on *Use of Tapered Longitudinal Joints such as the Notched Wedge Joint* for additional information.

Figure B-3 depicts the relationship between the lateral offset to the edge condition, its exposure to traffic, and when positive barrier should be considered. The use of the pavement drop-offs treatment guidelines along with Table B-1, Table B-2, Figure B-1, and Figure B-3, provides a practical approach to the use of positive barriers for the protection of vehicles from pavement drop-offs. Other factors, such as the presence of heavy machinery, construction workers, or the mix and volume of traffic, may make positive barriers appropriate, even when the edge condition alone may not justify the barrier.

NOTE: An approved end treatment should be provided for any positive barrier end located within the clear zone.



Notes:

(1) E = C x T

Where : C = portion of average daily traffic volume traveling within 20 feet (generally two adjacent lanes) of the edge of the dropoff condition.

T = duration time in years of the dropoff condition.

Figure B-3. Conditions Indicating Use of Positive Barrier.

Appendix C — Driveway Design Guidelines

Contents:

Section 1 — Overview

The purpose of this Appendix is to provide guidance on the design of driveway connections providing vehicular access to a highway on the state highway system. This Appendix governs the geometric design criteria for driveways.

Because field conditions are highly variable with respect to driveways, the guidance provided herein may not always be completely applicable. Therefore, departures from this design guidance for driveways to meet field conditions do not require or constitute a need for any type of design exception or design waiver as part of a State roadway project design plan; however, they may require support documentation in the form of traffic operations and safety analysis.

Additional information can also be found in TxDOT's *Access Management Manual* for permitting guidelines and for additional access discussion.

Section 2 — Introduction

General Guidelines

Driveways provide the physical transition between the public highway and the abutting property. Driveways should be located and designed to minimize negative impacts on traffic operations while providing safe entry and exit to / from the abutting property. The location and design of the driveway should take into account characteristics of the roadway, the abutting property, and the potential users. To ensure that driveways provide for safe and efficient traffic movements, it is necessary to consider the driveway's critical dimensions and design features. This Appendix applies to new driveways, and modification of existing driveways.

Definitions

- **Access Connection:** Facility for entry and/or exit such as a driveway, street, road, or highway that connects to a highway on the state highway system; as defined by the Texas Administrative Code Title 43 Part 4 Chapter 11 Subchapter C §11.51(1).
- **Commercial Driveway:** An entrance to, or exit from, any commercial, business, or similar type establishment.
- **Divided Driveway:** A driveway providing a raised or depressed median, between the ingress/egress sides of a driveway. Medians can be painted (fully traversable) when curbing is not allowed within the right-of-way, slightly raised curb (mountable) when U-turns are allowed or curbed (traversable) when U-turns are not allowed.
- **Driveway Apron**: On curb and gutter sections, that part of a driveway from the pavement to a selected point that is usually 6 inches in elevation above the edge of pavement (although it may vary by location or roadway) or to the right-of-way, whichever is greater. On sections with a drainage ditch, that part of a driveway from the edge of pavement to the right-of-way line.
- **Effective Turning Radius:** The minimum radius appropriate for turning from the right-hand travel lane on the approach street or driveway to the appropriate lane of the receiving street or driveway. This radius is determined by the selection of a design vehicle appropriate for the streets or driveway being designed and the lane on the receiving street or driveway into which that design vehicle will turn. Urban roadways with limited distance from the road to the right of way line may use lesser radii that fit within state right of way.
- **Farm/Ranch Driveway:** A Private Driveway providing ingress/egress for vehicles and farm/ranch equipment associated with the operation of the farm/ranch. Such driveways may also serve the residence of persons living and working on the farm/ranch and the other associated buildings.

- **Field Driveway**: A limited-use Private Driveway providing occasional/infrequent ingress/egress for equipment used for the purpose of cultivating, planting and harvesting or maintenance of agricultural land, or by equipment used for ancillary mineral production.
- **Non-simultaneous Two-Way Driveway**: A driveway intended to accommodate both entering and exiting traffic but not at the same time. For example, if an exiting vehicle is present in the driveway, the entering vehicle must wait until the exiting vehicle has cleared the driveway.
- **One-way Driveway**: A driveway designed for either an ingress or egress maneuver but not both.
- **Private Driveway:** An entrance to or exit from a residential dwelling, farm, or ranch for the exclusive use and benefit of the permittee; as defined by the Texas Administrative Code Title 43 Part 4 Chapter 11 Subchapter C §11.51(20).
- **Private Residential Driveway:** A Private Driveway serving a residential dwelling with anticipated P design vehicle and less than 20 vehicles per day using the driveway. A Private Residential Driveway should be designed as a Commercial Driveway if the anticipated design vehicle is SU or larger, or more than 20 vehicles per day are anticipated to use the driveway. Refer to AASHTO's *A Policy on Geometric Design of Highways and Streets* for information on the various design vehicle designations.
- **Public Driveway (Streets and Roads):** An approach from a publicly maintained street, road, or highway; as defined by the Texas Administrative Code Title 43 Part 4 Chapter 11 Subchapter C §11.51(21). Refer to Chapter 2, Section 9 - Off-System Roadways Intersecting Department Projects for design requirements of Public Driveways.
- **Radial Return or Flared Taper Return:** The physical connection transition geometry between the driveway and the roadway. A Radial Return is a curved radius. On a curb and gutter section, a Flared Taper Return may be a drop-down curb (parallel to the roadway) or an angular return curb (chorded between roadway and driveway). The top of curb profile must be 10 percent or flatter.
- **Service Driveway:** A Commercial, Public, or Private Driveway for occasional or infrequent use by vehicles or equipment to service an oil or gas well, electric substation, water well, water treatment plant, sewage lift station, waste water treatment plant, detention basin, water reservoir, emergency services, automated or remotely controlled pumping station, logging road, and other activities that may be identified by TxDOT.
- **Simultaneous Two-Way Driveway**: A driveway designed with a combination of return radius and throat width that allows a selected design vehicle to enter at the same time that another selected design vehicle is exiting the driveway.
- **Throat Length:** The distance parallel to the centerline of a driveway to the first on-site location at which a driver can make a right turn or a left turn; measured on roadways with curb and gutter, from the face of the curb, and on roadways without a curb and gutter, from the edge of the shoulder. Refer to Figure C-2.

- **Throat Width**: The driveway width measured at the end of the return radii. Refer to Figure C-2.

Section 3 — Driveway Design Principles

General Guidelines

The following guidelines apply to all driveways to a state highway.

- The driveway placement should be such that drivers approaching from the main roadway will have sufficient sight distance to ascertain the driveway's location in order to safely decelerate and complete the entry maneuver. Also, the driveway placement should be such that an exiting driver will have sufficient sight distance to judge a safe gap in oncoming traffic. Refer to Chapter 2, Section 3, Intersection Sight Distance for sight distance factors to be taken into consideration.
- For selecting appropriate driveway spacing distance or to determine if an acceleration or deceleration lane is warranted, refer to TxDOT's Access Management Manual. When acceleration or deceleration lanes are provided, refer to Chapter 3 for proper lengths.
- Each driveway should accommodate the effective turning radius of the appropriate design vehicle. The appropriate design vehicle will generally be the passenger car (AASHTO P design vehicle) unless the driveway will routinely be expected to handle more than four larger vehicles per hour. Examples of facilities for which a larger design vehicle would normally be appropriate include truck terminals, bus terminals, and connections that serve the loading docks of shopping centers. Figure C-1 illustrates the effects of the radius on the right-turn entry and exit maneuver.
- A Flared Taper Return is easier to construct than a Radial Return, but is less effective in terms of conforming to the turning path of a vehicle. The use of a Flared Taper Type Return should generally be limited to low volume driveways. When a curb and gutter Radial Return or Flared Taper Return encroaches on a pedestrian facility, a curb return consistent with TxDOT's Pedestrian Facilities Curb Ramps standard drawings must be provided.
- Figure C-2 illustrates the driveway design elements including return radius, entry width, exit width, throat width, and throat length.
- With the exception of private residential driveways, farm/ranch driveways, field driveways, and driveways that are designed and signed for one-way operation (i.e. ingress or egress only but not both), driveways should be designed to accommodate simultaneous entry and exit by the appropriate design vehicle.
- Driveways where pedestrian traffic is a potential, must be designed to maintain an accessible pedestrian route that is at least 4-ft wide across the driveway (see Section 6, Bicycle and Pedestrian Considerations).
- It is prohibited for private property owners to place markings or signs (such as Stop Signs, Entry or Exit signs) near driveways within the state right of way.

- ◆ Throat length is a part of the overall design and functioning of a driveway considering driveway geometry, frontage road and driveway traffic volumes over time, traffic patterns and movements, vehicle types, and property site layout.

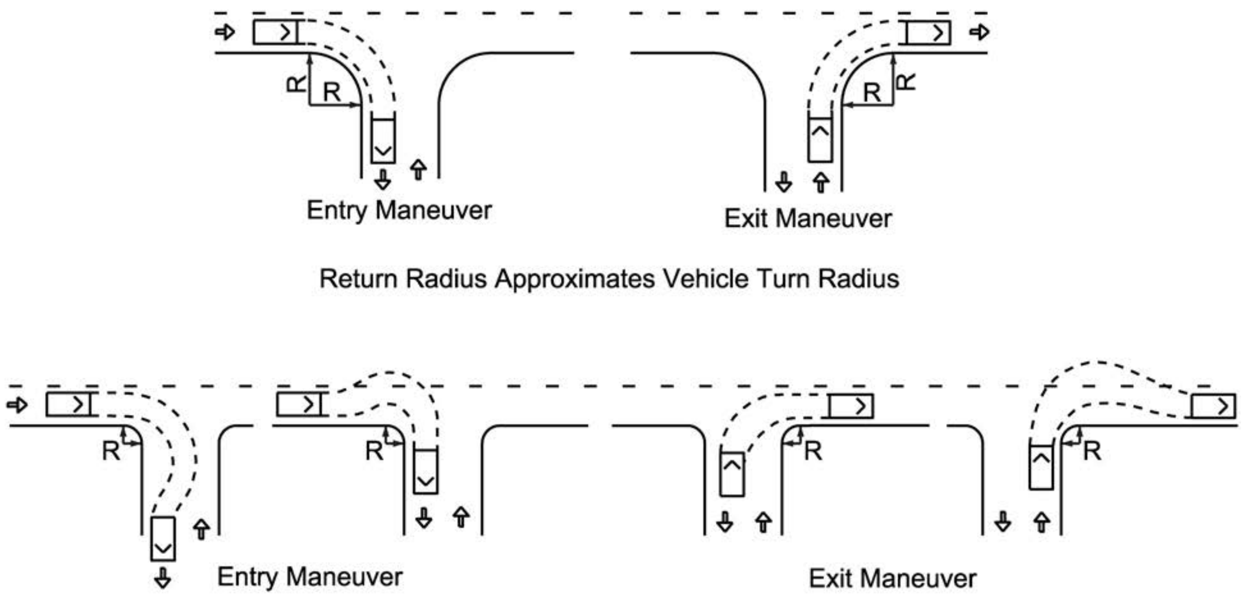


Figure C-1. Effects of Return Radius on the Right-Turn Maneuver

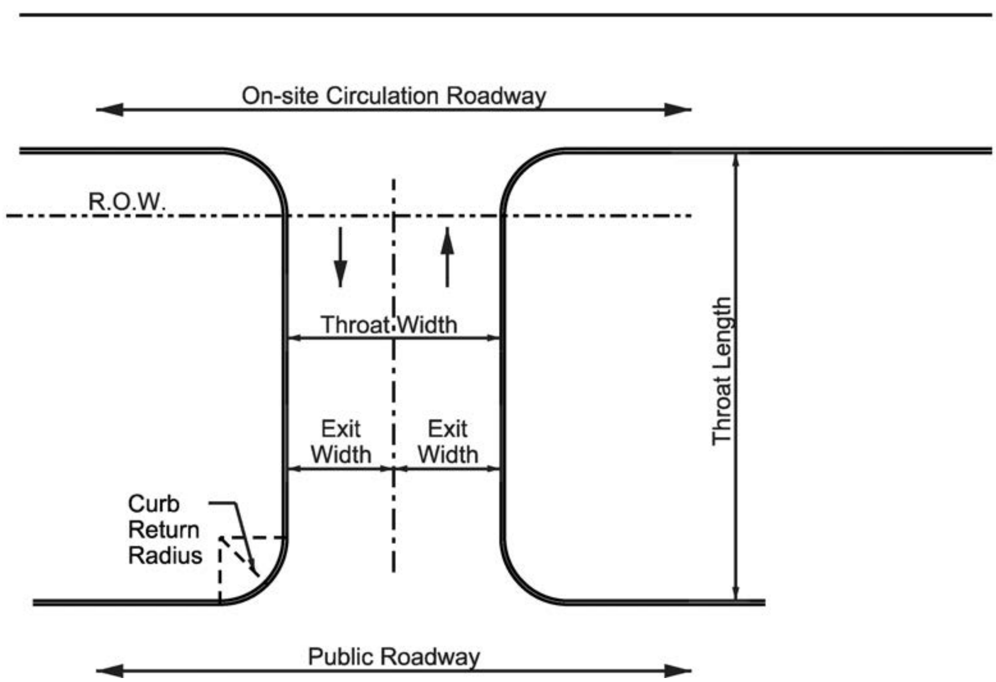


Figure C-2. Driveway Design Elements

Geometrics for Two-Way Driveways

The following are standards for two-way driveways.

- Private Residential Driveway are normally designed as non-simultaneous two-way driveways. Standard design criteria for private residential driveways are provided in Table C-1.

Table C-1: Design Criteria for Two-Way Residential Driveways

Radius (ft)	Throat Width Standard (ft)	Throat Width Maximum (ft)
15	14	24

NOTE: Urban roadways with limited distance from the road to the right of way line may use lesser radii that fit within state right of way. Some residential driveways may have wider throats due to unusual site conditions.

- Commercial Driveways should be based on the appropriate design vehicle. Properties served by such driveways include, but are not limited to, truck stops, warehouses, concrete batch plants, sources of aggregate, RV sales/truck sales and RV parks. The design should also consider future roadway traffic and local conditions and incorporate simultaneous two-way driveways if justified.
- Two exit lanes are recommended when the expected driveway exit volume exceeds 200 vph.
- In cases where one-way operation is appropriate, a condition of the driveway permit should require that appropriate one-way signing be installed and maintained.
- Table C-2 provides standard design criteria for two-way commercial driveways that would be expected to accommodate only P and SU design vehicles. Driveway designs for larger vehicles will be considered on a case by case basis.

Table C-2: Designs for Two-Way Commercial Driveways

Condition	Radius (R) (ft) (Min.)	Throat Width (W) (ft)
One entry lane and one exit lane, fewer than 4 SU vehicles per hour (see Fig. C-3)	25	28
One entry lane and one exit lane, 4 or more SU vehicles[3] per day (see Fig. C-3)	30	30
One entry lane and two exit lanes, without divider (see Fig. C-4)	25	40
One entry lane and two exit lanes, with divider (see Fig. C-5)	25	44[1]-55[2]
Two entry lanes and two exit lanes, with divider (see Fig. C-6)	25	56[1]-67[2]

Table C-2: Designs for Two-Way Commercial Driveways

Condition	Radius (R) (ft) (Min.)	Throat Width (W) (ft)
Notes: 1. See Table C-3 for minimum divider widths. 2. See Table C-3 for maximum divider widths. 3. Driveway designs for larger vehicles will be considered on a case by case basis. 4. Urban roadways with limited distance from the road to the right of way line may use lesser radii that fit within the state right of way.		

- On existing roadway reconstruction projects, driveways may be reconstructed at the same width and location. However, all driveways being considered for reconstruction should be re-evaluated to determine if more appropriated widths, locations, or driveway deletions or changes will provide additional traffic safety and functionality. The potential impacts to the whole adjacent property due to a material change of access should also be considered. Refer to TxDOT's *Access Management Manual* for additional information.
- Service Driveways – Service driveways should be designed considering the vehicle type and frequency of use, current and future traffic operations on the state highway, and other local conditions.
- Field Driveways – The distance from the edge of the shoulder to a gate should be sufficient to accommodate the longest vehicle (or combination of vehicles such as a truck and trailer) expected. At a minimum, this will normally be a truck with trailer.
- Farm/Ranch Driveway – A typical design for a farm/ranch driveway should provide 25-ft return radii and a 20-ft throat width. The distance from the edge of pavement must be sufficient to store the longest vehicle, or combination of vehicles, expected. At a minimum, this will normally be a truck with trailer.

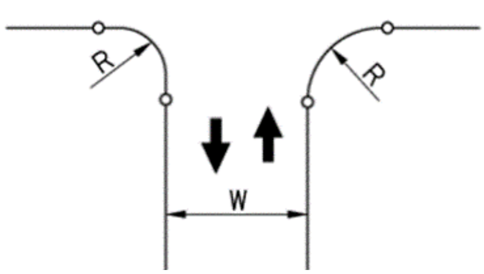


Figure C-3. One Entry Lane/One Exit Lane

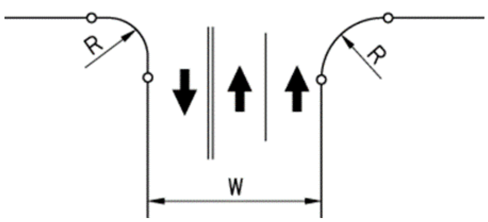


Figure C-4. One Entry Lane/Two Exit Lanes (Without a Divider)

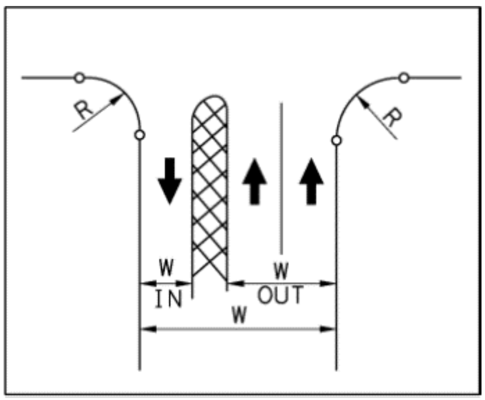


Figure C-5. One Entry Lane/Two Exit Lanes (With a Divider)

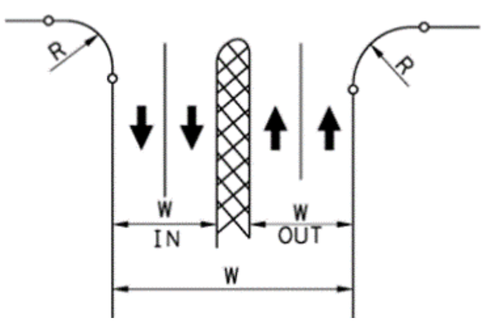


Figure C-6. Two Entry Lanes/Two Exit Lanes (With a Divider)

Divided Driveways

A raised or depressed separation between the entry and exit sides of a divided driveway needs to be visible to drivers. Suggested treatments and divider sizes are shown in Table C-3:

Table C-3: Dimensions for Dividers in the Driveway Throat to Separate Entry and Exit Sides of the Driveway

Treatment	Width[3] (ft)	Length (ft)
Slightly raised[1] (4-in) with contrasting surface	4[2] –15	20

Notes:

1. For Rural - Rounded edges, 30^{o} to 45^{o} slope. (See Figure C-7).
2. 6-ft minimum for pedestrian refuge, measured from back of curb (desirable) or from nominal face of curb (minimum).
3. Measured from nominal face of curb.

Figure C-7 illustrates a slightly raised divider (height ≥ 4 inches).

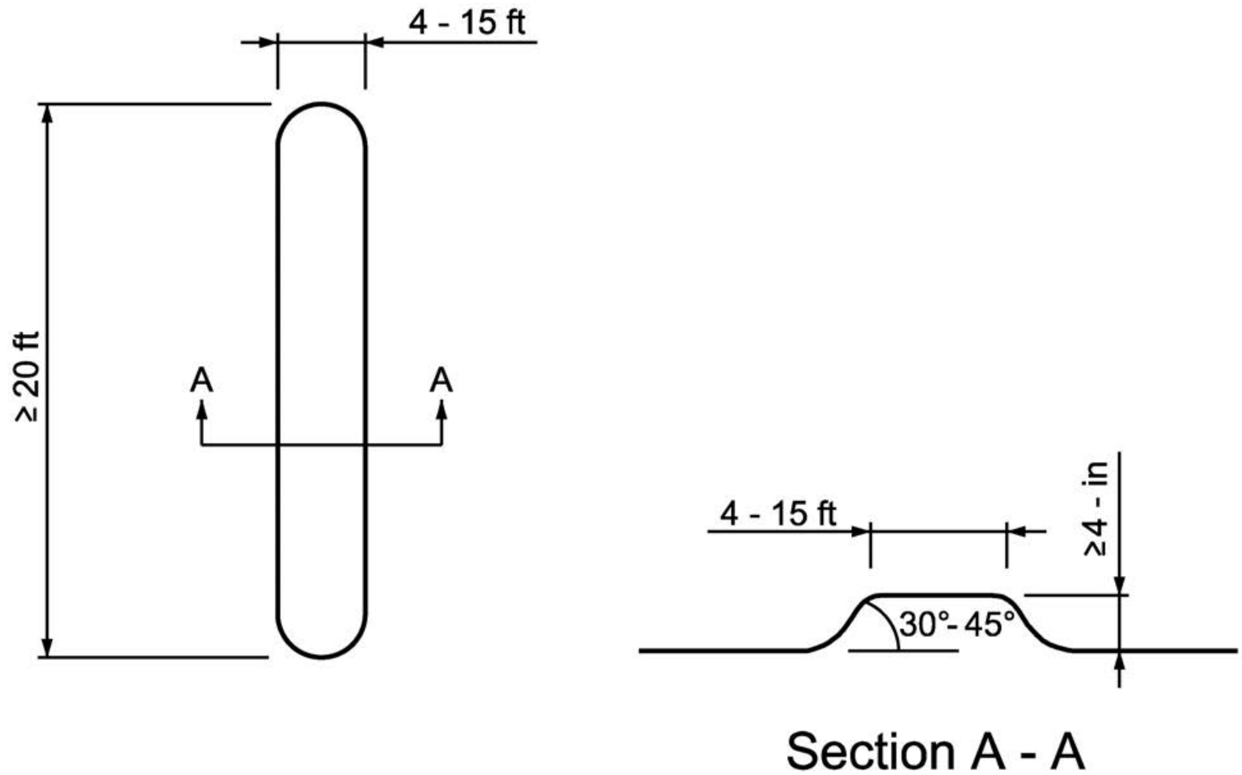


Figure C-7. Illustration of Slightly Raised Divider

A divided driveway is desirable in the following situations:

- There are a total of four or more entering and exiting lanes; or
- A large number of pedestrians (30 or more in a one-hour interval) routinely cross the driveway.

Locating appropriate signing (reference TMUTCD) and lighting within a divider may assist approaching drivers in determining the driveway's location and geometrics.

An excessively wide divider may confuse drivers and cause them to think there are two closely spaced, two-way driveways. To avoid this problem, the recommended maximum width of a divider is 15-ft. On the other hand, a divider that is too narrow may not be adequately visible to the motorist. Therefore the recommended minimum width of a slightly raised divider (height ≥ 4-in) is 4-ft, and 6-ft for a Pedestrian Refuge.

Section 4 — Profiles

Public driveways and commercial driveways should be constructed with a vertical curve between the pavement cross-slope and the driveway approach and between changes in grade within the driveway throat length. A private residential driveway may be constructed without vertical curves provided that a change in grade does not adversely affect vehicle operations. Typically, a change in grade of 3 percent or less and a distance between changes in grade of at least 11-ft accommodates most vehicles. However, literature suggests that a 6 percent to 8 percent change in grade may operate effectively. Individual site conditions should be evaluated to accommodate the vehicle fleet using the driveway. Refer to AASHTO's *A Policy on Geometric Design of Highways and Streets* and NCHRP Report 659 *Guide for the Geometric Design of Driveways* for additional information.

Driveway Grades

To achieve satisfactory driveway profiles, some of the significant factors to be considered are:

1. Abrupt grade changes, which cause vehicles entering and exiting driveways to move at extremely slow speeds, can create:
   - The possibility of rear end collisions for vehicles entering the driveway, and
   - The need for large traffic gaps that may be unavailable or infrequent, causing drivers to accept inadequate gaps.
2. Where a driveway is expected to cross a pedestrian or bicycle facility now or in the future, special design requirements may apply. Refer to Chapter 6, Section 4 – Bicycle Facilities and Chapter 7, Section 3 – Pedestrian Facilities for more information.
3. The comfort of vehicle occupants and potential vehicle damage (i.e. prevent the dragging of center or overhanging portion of passenger vehicles).
4. Grades must be compatible with the site requirements for sight distance and drainage to prevent excessive drainage runoff from entering the roadway or adjacent property.

Because a large combination of slopes, tangent lengths, and vertical curves will provide satisfactory driveway profiles, some generalizations should be considered relative.

On curb and gutter sections, placement of vertical curves should be at the projected gutter line and not closer to the travel lanes unless curb and gutter returns and proper drainage are provided. The entire curb and gutter for the length of the curb cut should be removed and the gutter pan recast as an integral part of the driveway apron. When a curved radius is used, the curb and gutter should continue through the radius return. No gap in the curb and gutter should be allowed withing the radius return unless the gap is provided to accommodate a curb ramp when crossing a bicycle or pedestrian facility.

Profile vertical curves may not be practical for urban roadways with short driveway lengths.

As shown in Table C-4, the suggested maximum changes in driveway grades with a vertical curve (between the pavement cross slope and the driveway apron slope) are approximately 10 percent for private residential driveways and approximately 8 percent for all other driveways.

Table C-4: Suggested Maximum Change in Grade with a Vertical Curve

Driveway	Change in Grade (A)[1]
Private Residential Driveways	10%
All Other Driveways	8%
Notes: 1. Change in grade between the pavement cross-slope and the driveway apron slope	

Construction practice can provide a suitable sag vertical curve between the pavement cross-slope and the driveway apron when the apron length L_a (see Figure C-8) is greater than or equal to 20-ft.

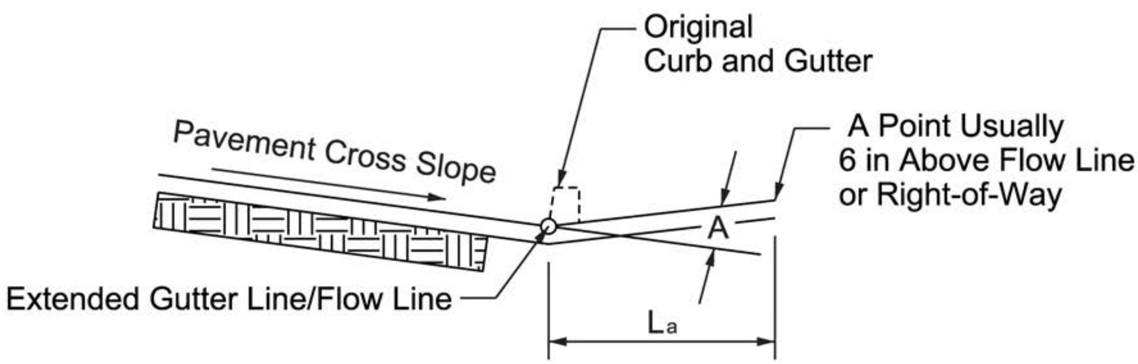


Figure C-8. Suggested Dimensions to Achieve an Appropriate Vertical Curve

Where possible, the driveway grade should be limited to 6 percent or less within the roadway right-of-way. Maximum driveway grades are up to 12 percent for private residential driveways and up to 8 percent for other driveways. Other driveway grades may be considered on a case by case basis.

The length of the vertical curve between the pavement cross-slope and the driveway apron is a function of the algebraic difference in the grades. Table C-5 provides the desirable and minimum lengths for these vertical curves.

Table C-5: Length of Vertical Curve L (feet) For a Change in Grade Between the Pavement Cross-Slope and the Driveway Apron Slope

Change in Grade, A	Crests		Sags	
	Des.	Min.	Des.	Min.
	ft	ft	ft	ft
4-5%	5	3	7	4
6-7%	6	4	8	5
8-10%	8	5	10	7

The length of the vertical curve at other points of driveway grade change is also a function of the algebraic difference in the grades. Table C-6 provides the typical lengths for these vertical curves.

Figures C-9 through C-11 illustrate typical driveway profiles.

Table C-6: Typical Length of Vertical Curve, L, For Change in Grade in Driveway Profile

Change in Grade, A	Crest		Sag	
	Private Residential Driveways	Other Driveways	Private Residential Driveways	Other Driveways
	ft	ft	ft	ft
4-5%	2	5	3	6
6-7%	3	5	5	7
8-10%	4	6	6	8

A minimum K-value of 4 ($K = L/A$) is recommended for driveways accommodating low ground clearance and / or long wheelbase vehicles (e.g., articulated beverage trucks, car haulers, and passenger car-trailer combinations).

Profiles on Curb and Gutter Sections

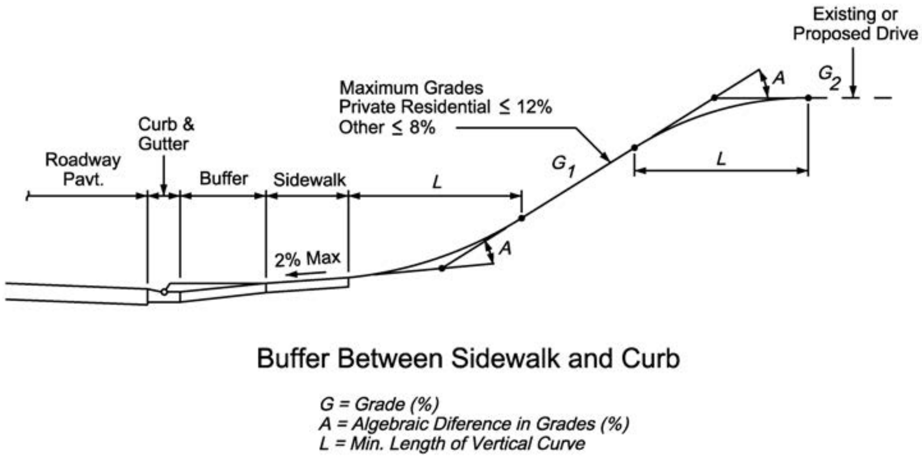


Figure C-9. Roadway with Curb and Gutter, Driveway Profiles on an Upgrade

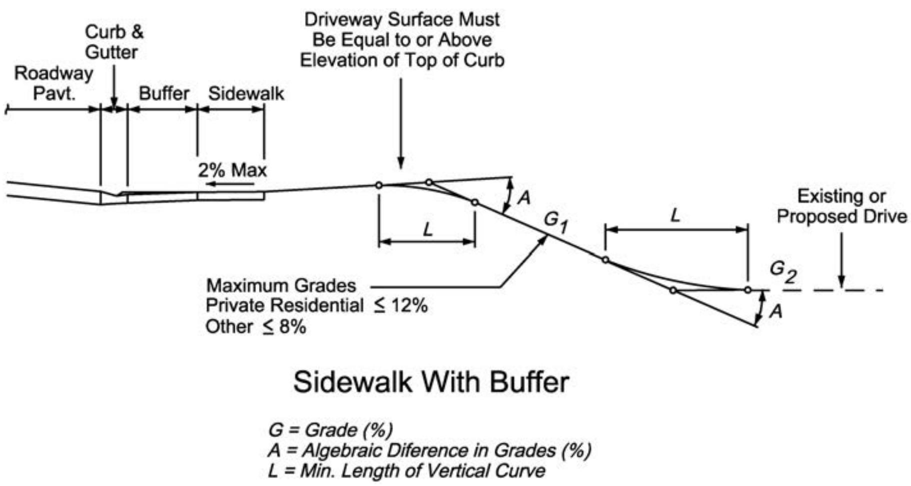


Figure C-10. Roadway with Curb and Gutter, Driveway Profiles on a Downgrade

See Tables C-5 and C-6 for lengths of vertical curves.

Profiles with Drainage Ditch

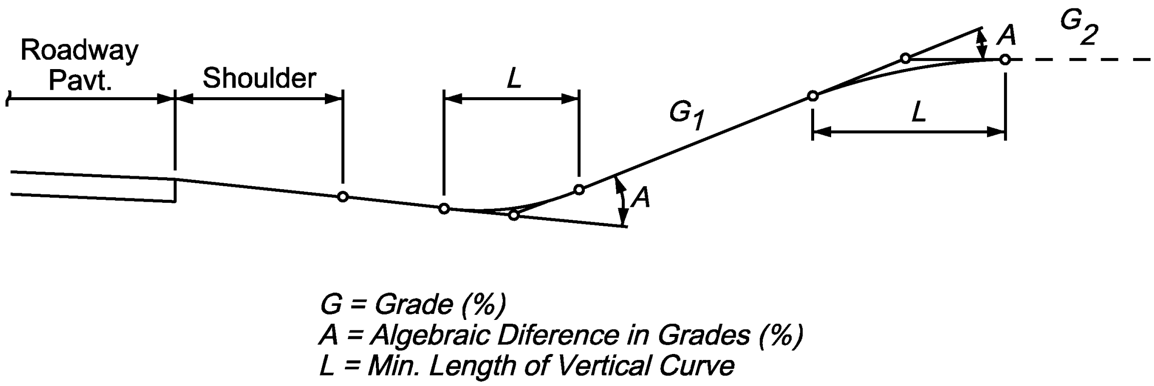


Figure C-11. Driveway Profiles on Roadway with Drainage Ditch

See Tables C-5 and C-6 for lengths of vertical curves.

Section 5 — Driveway Angle

Two-Way Driveways

Two-way driveways should intersect the roadway at an angle of ninety degrees unless it is determined that a lesser angle will provide satisfactory traffic operations for the highway. Just as it undesirable for two roadways to intersect at highly skewed angles, it is undesirable for most driveways to intersect the roadway at a large skew. When a skew angle forces drivers to deal with a turning angle that is much less than or greater than 90 degrees, drivers will have greater difficulty turning their heads to scan the through roadway for an adequate gap, and more distance and time is required to complete an acute angle turning movement. Research has shown that the intersection angle of driveways should not be skewed from 90 degrees by more than 15 to 20 degrees. Suggested limiting values of two-way driveway angles are:

- **Private Residential Driveway:** 75 degrees;
- **Commercial Driveway:** 75 degrees; A commercial driveways expected to have a volume of 400 vehicles per day or two or more trucks/large vehicles in a one-hour period must be designed as a public driveway;
- **Public Driveway, Service Driveway and Field Driveway:** 80 degrees.

One-Way Driveways

The angle of intersection between the centerline of a one-way driveway and the edge of pavement of the public roadway may be between 45 and 90 degrees. Engineering judgement should be used when selecting an intersection angle for one-way driveways based on field conditions and sight distance viewing angles (i.e., right-turn entry-only, right-turn exit-only, left and right-turn entry-only, or left and right-turn exit-only).

Section 6 — Bicycle and Pedestrian Considerations

General Guidelines

Where a driveway is expected to cross a bicycle or pedestrian facility now or in the future, special design requirements may apply.

Driveway crossings should be designed so that pedestrians, cyclists, and drivers are able to negotiate the driveway crossing efficiently and safely. Desirably, the passable portion of the driveway should be delineated within the limits of the driveway (e.g. construction joints or visual contrast across the driveway). Refer to Chapter 6, Section 4 – Bicycle Facilities and Chapter 7, Section 3 – Pedestrian Facilities for more information.

Section 7 — Visibility

Drivers must be able to locate a driveway in time to reduce speed and negotiate the entry maneuver. Lighting can be used to alert drivers to driveway opening locations a considerable distance in advance by illuminating the junction of the driveway and the highway. On curbed facilities, the use of vegetation buffers between the back of curb and sidewalk can help define driveway edges and make the driveway location more discernible for drivers, cyclists, and pedestrians.

On divided driveways, signing should be located within the divider separating the entrance and exit sides of the driveway and must take into consideration the sight distance for vehicles entering and exiting the driveway and cyclists and pedestrians crossing the driveway. Private signs and lighting are prohibited on state right of way.

Section 8 — References

1. A Policy on the Geometric Design of Highways and Streets, American Association of Highway and Transportation Officials.
2. Transportation and Land Development, Institute of Transportation Engineers, 2002.
3. TxDOT Access Management Manual.
4. Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right of Way, July 2011.
5. R. J. Jaeger, Guidelines for the Investigation and Remediation of Potentially Hazardous Bicycle and Pedestrian Locations, Traffic Engineering and Safety Management Branch, North Carolina Department of Transportation, August 2003.
6. Charles V. Zegeer, Cara Seiderman, Peter Lagerway, Mike Cynecki, Michael Ronkin and Robert Schneider, "Pedestrian Facilities Users Guide – Providing Safety and Mobility," Publication No. FHWA No. FHWA-RD-01-102, Federal Highway Administration, US Department of Transportation, March 2002.
7. NCHRP Report 659 Guide for the Geometric Design of Driveways, Transportation Research Board of the National Academies, 2010.

Appendix D — Right-Turn Slip Lane Design Guidelines

Contents:

Section 1 — Introduction

Right-turn slip lanes are advantageous to motorists as they reduce delays by separating right-turning traffic from through lanes. The right-turn slip lane channelizing island can also provide a refuge area for crossing pedestrians, reducing their exposure by allowing them to cross the roadway in two stages. Providing safe mobility for pedestrians and cyclists while reducing delays for motorists are potentially conflicting objectives. Accordingly, right-turn slip lane designs should create a balance between the safety and mobility of all roadway users.

This Appendix provides guidance on the design of right-turn slip lanes, including lane and raised island geometric layouts, pavement marking guidelines, pedestrian and bikeway guidelines, and accommodations for pedestrians with disabilities. Guidance is provided for urban, suburban, and rural roadway environments. Common issues at existing right-turn slip lanes have also been identified and treatment options for retrofitting are provided.

Section 2 — New Construction

Urban Design

Right-turn slip lanes at urban intersections shall be designed to accommodate both motorists and pedestrians. Considerations should be given for crosswalk compliance by motorists and pedestrians, design vehicle accommodations, speed of turning traffic, provisions of auxiliary lanes, adjacent land uses, visibility of anticipated pedestrian traffic, and acuity of the cross-street traffic from the right turn drivers' perspective. In areas where pedestrian activity is moderate to high, raised crosswalks may be installed to slow turning motorists and improve their likelihood of stopping and yielding to crossing pedestrians. A raised crosswalk may in some instances have the benefit of reducing the length of a required ramp on the pedestrian island. However, raised crosswalks are not recommended along high-speed facilities.

The following recommendations address these considerations and are reflected in the configurations shown in Figure D-4 (with a deceleration lane) and D-5 (without a deceleration lane).

1. **Angle of Entry:** The angle of entry between the slip lane and the cross street is recommended to be 70 degrees. This configuration slows motorists, reduces the head-turning movement to look for gaps in oncoming traffic, and makes it easier for motorists to identify crossing pedestrians. If an angle of 70 degrees is not achievable due to constraints, reduce accordingly. The minimum recommended angle is 55 degrees.
2. **Curb Radii and Curb-to-Curb (throat) Width:** The majority of traffic on urban streets is expected to be passenger cars and single unit trucks. However, to accommodate the turning movement of larger vehicles, the curb radius and curb-to-curb width can be designed for a larger design vehicle while striped to delineate the path for a smaller vehicle. For guidance on curb radii design for different vehicle classes, see the discussion in Chapter 3 on *Urban Streets* and the *Intersections* subsection, and in *Minimum Designs for Truck and Bus Turns* in Chapter 7. See Signage and Pavement Markings below for lane markings.
3. **Channelizing Island:** Refer to Chapter 7, *Miscellaneous Design Elements*, Section 7, *Minimum Designs for Truck and Bus Turns - Channelization* for proper size of channelizing islands. Islands are recommended to have a minimum side length of 15-ft, excluding the corner radii, as discussed in AASHTO's *A Policy on the Geometric Design of Highways and Streets*. Channelizing islands should be offset from the edge of the traveled way to reduce their vulnerability. A 12-ft side length may be used in special circumstances, where the 15-ft minimum can't be met due to highly constrained conditions. See Figure D-1 for design guidance on curb offset and tapering. In the presence of a bicycle lane, which serves as a separation between the curb and the travel lane, curbs need not be offset. Additional information on appropriate curb type and design can be found in Chapter 2, *Cross Sectional Elements*, and in Chapter 7, *Minimum Designs for Truck and Bus Turns*.

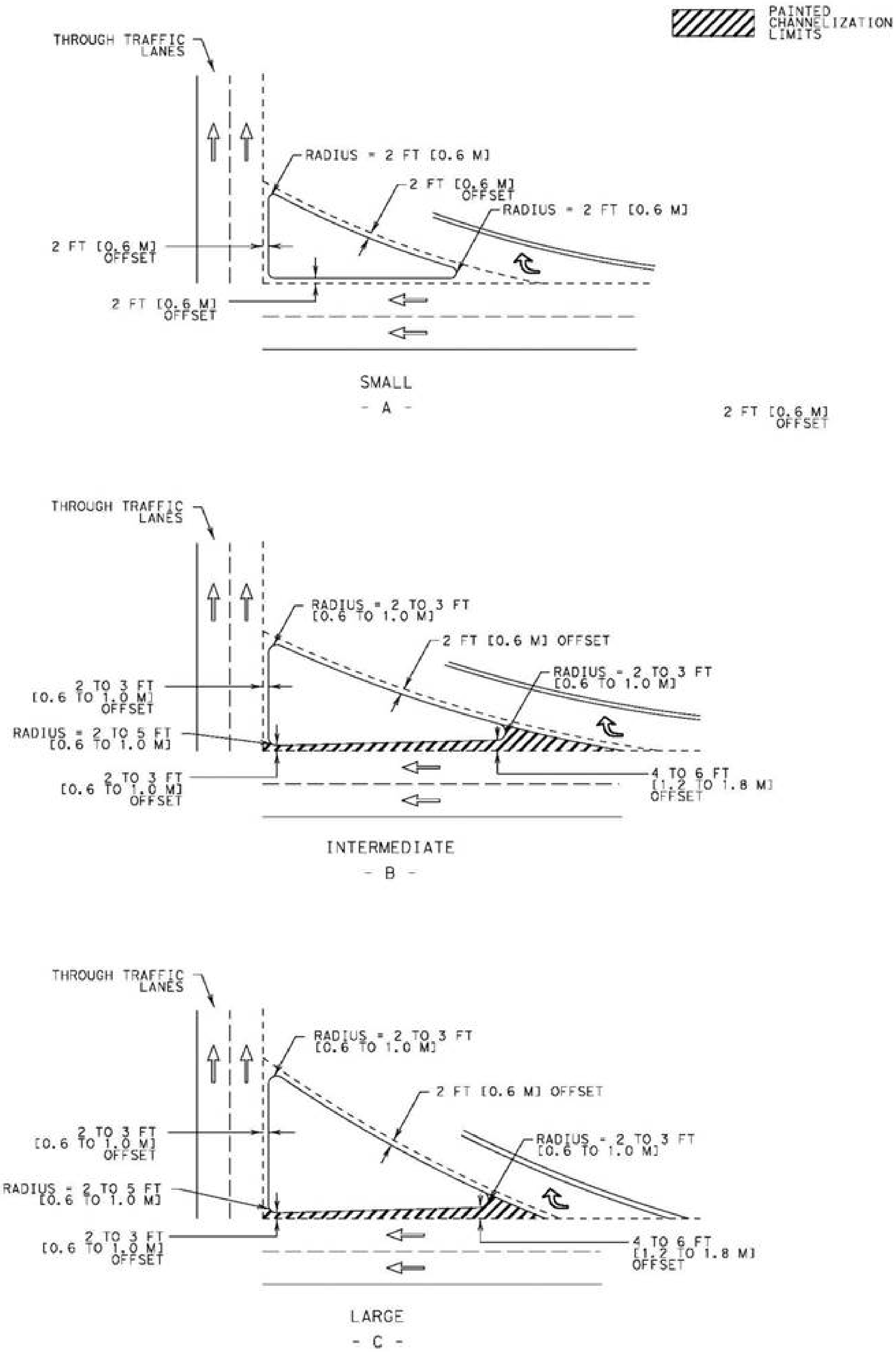
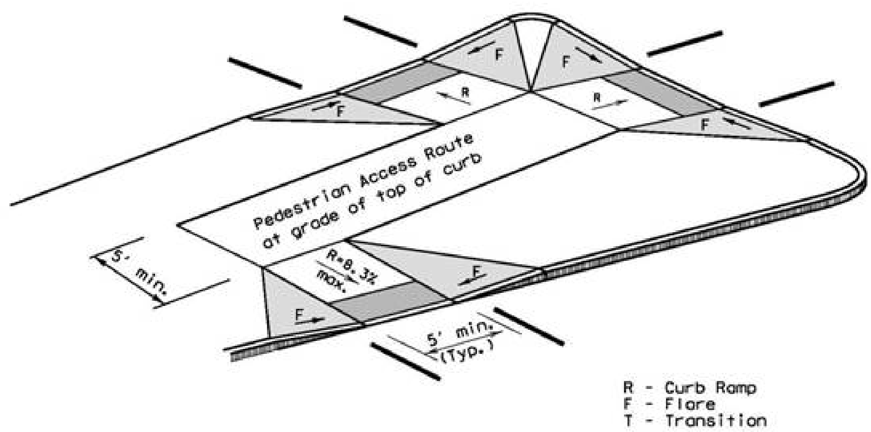


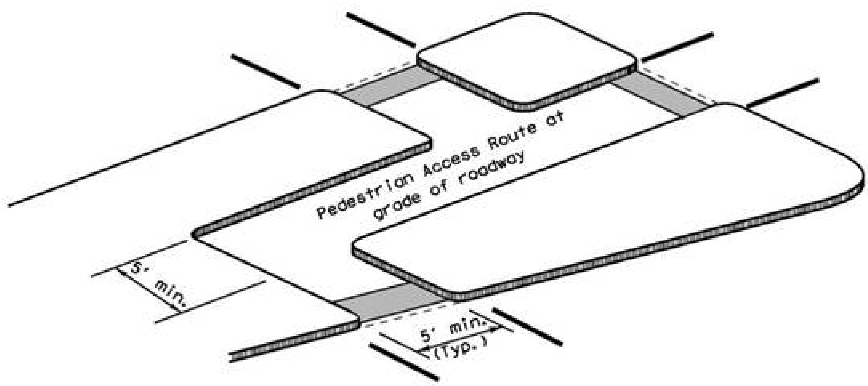
Figure D-1. Details for Channelizing Island Design for Right-Turn Slip Lane

Pedestrian accommodations are a central component of the design. The pedestrian access routes across channelizing islands are typically set at the top of curb elevation using a series of curb ramps and landings (Figure D-2-A). Smaller channelizing islands may not have adequate space to provide the necessary curb ramps, so the pedestrian access route can be cut through the island flush with the gutter grades of the roadway (Figure D-2-B). This option may be easier for some pedestrians to navigate and does provide some wayfinding benefits but it can also collect water and debris, increasing maintenance needs. A third option is to raise the accessible route to at least 2-in above roadway gutter grade (Figure D-2-C). This solution helps reduce the maintenance required for cut-through islands and still provides some wayfinding clues. If a raised crosswalk is used, this would also result in a reduction in the length of the needed ramp for the options depicted in Figures D-2-A and D-2-C. Provide flared sides where the pedestrian circulation path crosses the curb ramp. Flared sides shall be sloped at 10 percent maximum, measured parallel to the curb. Curb returns may be used only where pedestrians would not normally walk across the ramp because the adjacent surface is planted, substantially obstructed, or otherwise protected. All components of the accessible route should be constructed at a minimum 5-ft width to provide adequate room for pedestrian passage.



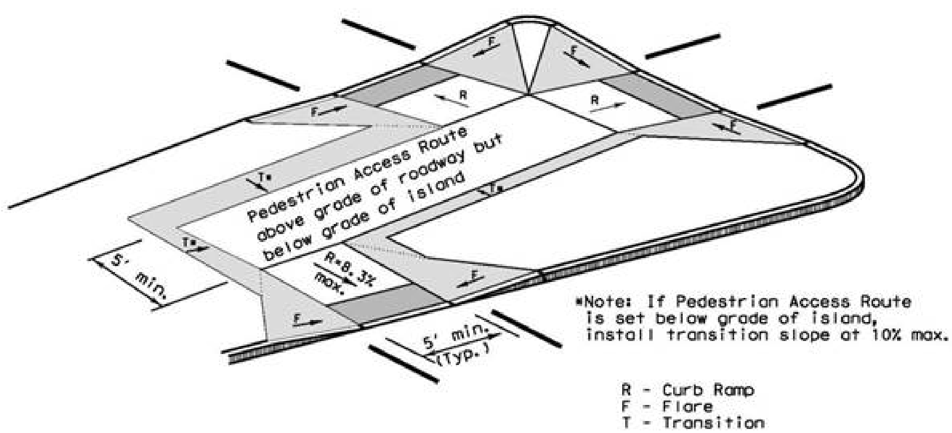
ACCESS ROUTE ACROSS TOP OF ISLAND

- A -



ACCESS ROUTE CUT THROUGH ISLAND

- B -



ACCESS ROUTE PARTIALLY CUT THROUGH ISLAND

- C -

Figure D-2. Combination Island Ramps (Per Pedestrian Facilities Curb Ramp Standards)

If partial cut-through sections are used, a 10 percent flare and rounded corners along the pedestrian pathway should be used to provide better wheelchair mobility.

4. **Deceleration Lane:** These lanes allow motorists to decrease speed before negotiating a turn while separated from through traffic. This separation helps pedestrians identify right-turning vehicles. See *Number, Location, and Spacing of Access Connections* in Chapter 2 of the TxDOT *Access Management Manual* for volume thresholds for installing deceleration lanes. Refer to *Urban Streets* in Chapter 3 for design recommendations for deceleration lanes. In the event that conditions do not necessitate a deceleration lane or right-of-way is restricted, consideration should be given to using a taper, as defined in Chapter 3. See Figures D-4 and D-5 for sample right-turn slip lane designs with and without a deceleration lane.

5. **Acceleration Lane:** Acceleration lanes typically are not used on urban streets since:

   a. They make it more difficult for pedestrians, especially the visually impaired, to cross the turning roadway.

   b. Cross street drivers do not expect merging traffic.

   c. Acceleration lanes and driveways create unneeded conflicting movements between the driveway and acceleration lane.

   Accordingly, acceleration lanes are not advisable where pedestrian activity is anticipated.

6. **Drainage:** Any necessary inlets should be designed and placed on the upstream side of the crosswalk at a location that prevents—or limits to the extent practical—the spread of water into the crosswalk. Cut-through access should be situated as to minimize paths for water flow. Avoid placing drainage low points at or near the ADA curb ramps.

7. **Lighting:** Intersections with channelization should be illuminated. Lighting helps motorists identify islands, diverge and merge locations, turning roadways, and pedestrian crossings. Adequate lighting at urban intersections, including illumination of crossing locations, is important—particularly where pedestrian activity is expected at night.

8. **Apparatus and Pole Placement:** ITS equipment, signal and utility poles and apparatuses should be outside of paved pedestrian walkways and landing areas. Refer to the *Texas Manual on Uniform Traffic Control Devices (TMUTCD)* and TxDOT standard drawings for guidance on mounting heights and limits on object protrusion into pedestrian facilities. When pedestrian facilities are not initially installed, care should be taken to avoid the placement of apparatuses and poles in anticipated or planned locations of future pedestrian walkways and landing areas.

9. **Crosswalk Location:** Crosswalks should be placed toward the middle of the channelized island with a minimum of 20-ft between crosswalk and yield line for the intersecting street. Crosswalks may be placed near the beginning of the channelized island if conditions do not permit a centralized location or it is more conducive to the natural pathway of pedestrians. When the crossing is located at the beginning of the channelized island, care should be taken to place it such that there is enough space available at the ramp location for an appropriate landing area. Placement of the crosswalk near the end of the turning roadway is not recommended as motorists are expected to encroach on the crosswalk as they yield to oncoming traffic. Also,

motorists arriving at the downstream end of the turning roadway typically focus their attention on cross street traffic rather than crossing pedestrians.

10. **Crosswalk Orientation:** The pedestrian crosswalk should be oriented perpendicular to the turning roadway to shorten the crossing distance for pedestrians and to place approaching vehicles in the periphery of pedestrians.

11. **Crosswalk Markings:** At locations where pedestrian activity is anticipated, a "High-Visibility Longitudinal Crosswalk" pattern is recommended to delineate the crossing location. The transverse markings facilitate wayfinding for visually impaired pedestrians and the inclusion of longitudinal lines provides additional visibility for approaching motorists. Alternatively, longitudinal markings alone can be installed to define the crossing path. Refer to the *TMUTCD* for further guidance on the installation of crosswalk markings.

12. **Signage and Pavement Markings:** Yield signs are typically the appropriate control devices for right-turn slip lanes at urban intersections. The yield line is used alongside the yield sign to draw attention to the need to yield to cross-street traffic. Refer to the *TMUTCD* and TxDOT standard drawings for guidance on yield sign and yield line placement. Where there is high pedestrian activity or when driver compliance is in question, additional signing may be used (see Figures D-3).

Figure D-3. Supplemental Crosswalk Signs
W11-2 and W16-7PL Sign and Plaque (TMUTCD)

The travel lane should be striped to a minimum of 10-ft in width (11-ft typical) to accommodate a passenger vehicle, and the void area (shoulder) may be delineated by diagonal lines as shown in Figures D-4 and D-5. A raised truck apron may also be considered in the void area to further enhance the channelization of passenger vehicles.

13. **Bicycle Lane:** When a bicycle lane is used, it should be striped appropriately to define right-of-way and shared spaces as discussed in the AASHTO *Guide for the Development of Bicycle Facilities*, as shown in the *TMUTCD*, and as detailed on TxDOT standard drawings for bicycle lanes. Bicyclists intending to make a right turn can use the right-turn lane/turning roadway and operate like a motorized vehicle.

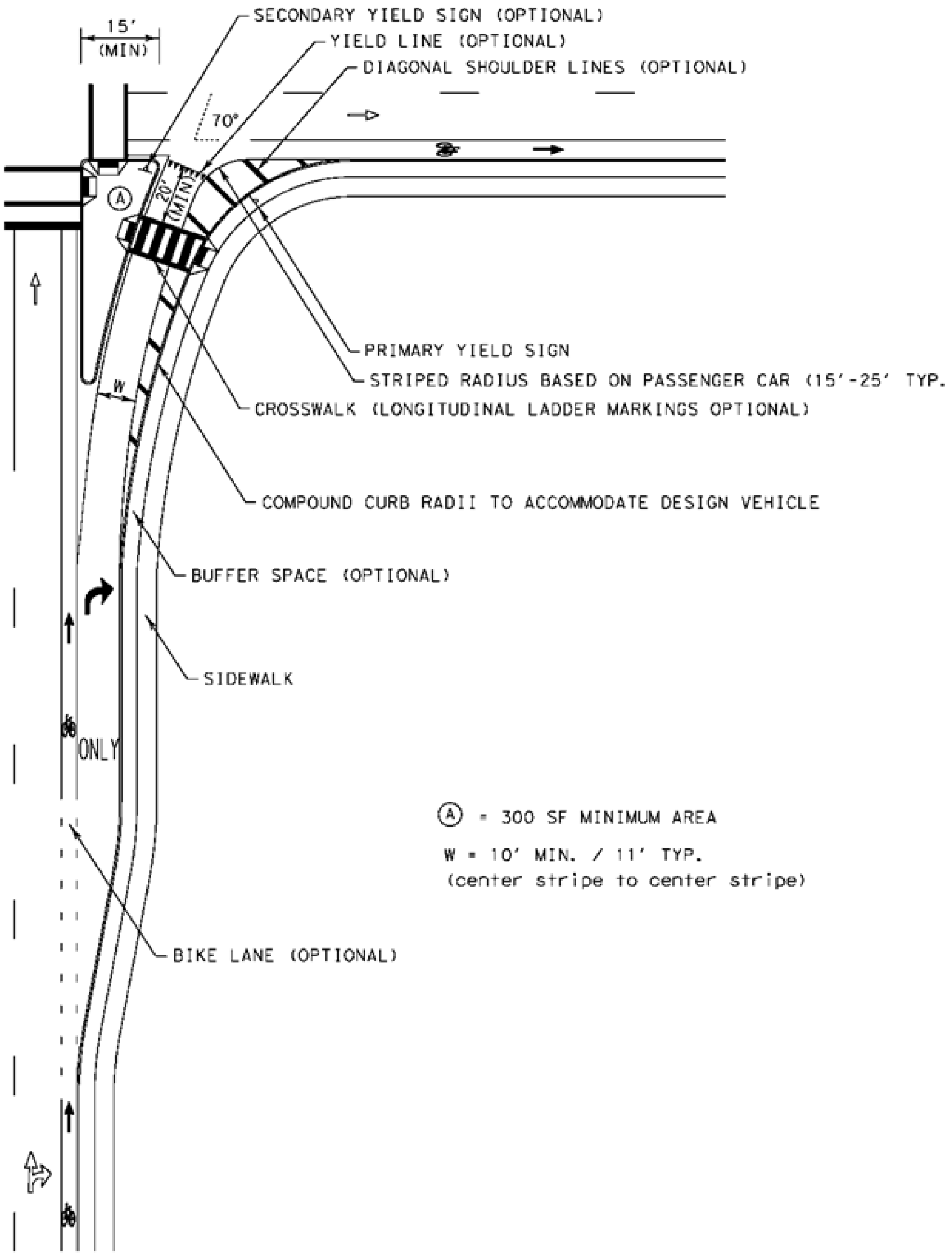


Figure D-4. Right-Turn Slip Lane Design for Urban Intersections with Deceleration Lane

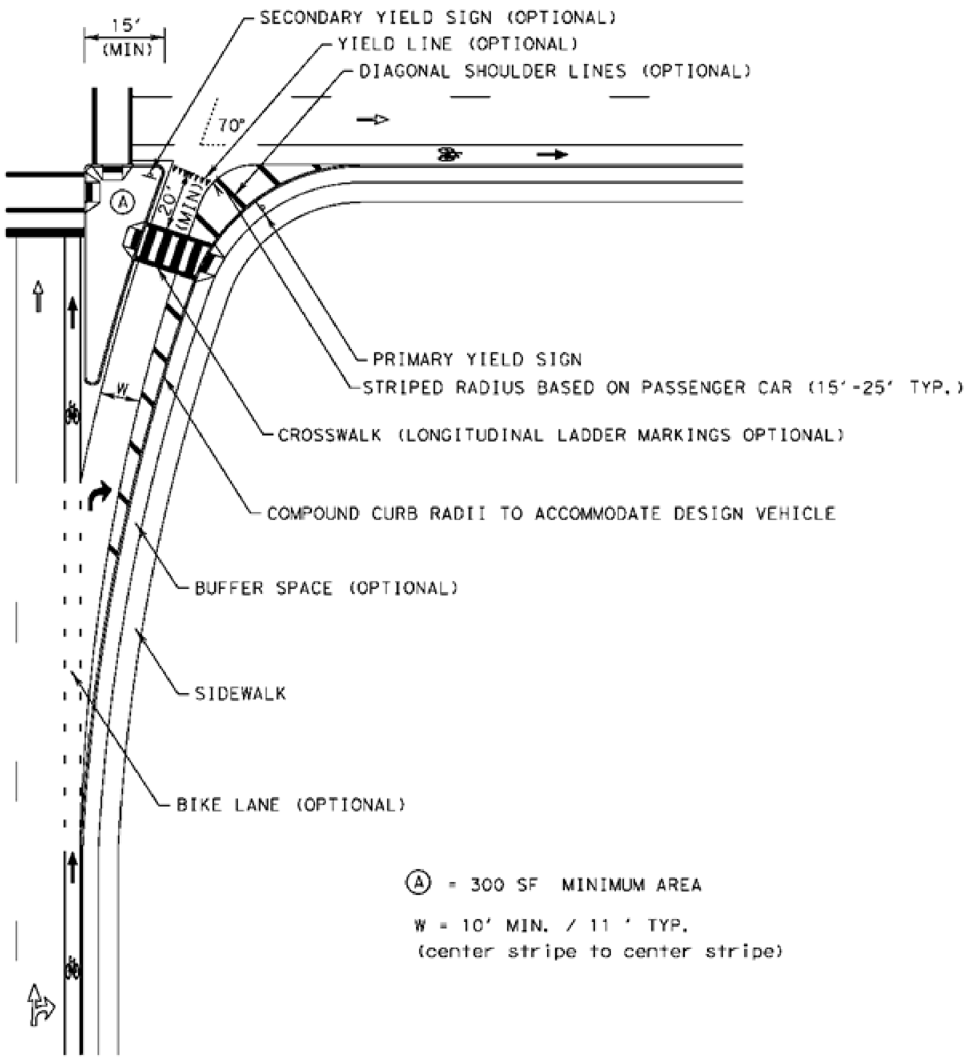


Figure D-5. Right-Turn Slip Lane Design for Urban Intersections without Deceleration Lane

Suburban Design

Pedestrian activity on suburban roadways tends to be in the range of light to moderate. The following recommendations address the presence of pedestrians and facilitate potential future retrofits without heavily impacting mobility.

1. **Angle of Entry:** See guidance provided under *Urban Design, Angle of Entry*.
2. **Curb Radii and Curb-to-Curb (Throat) Width:** The radii and throat width for right-turning roadways in suburban areas should be designed to accommodate larger vehicles. In the event that the area becomes more urbanized in the future, the turning roadway can be striped to delineate a tighter radius, promoting lower speeds and improving visibility of pedestrians for motorists (see Figures D-4 and D-5). See *Cross Sectional Elements* in Chapter 2 for striped lane width.
3. **Channelizing Island:** See guidance provided under *Urban Design, Channelizing Island*.
4. **Deceleration Lane:** See guidance provided under *Urban Design, Deceleration Lane*.
5. **Acceleration Lane:** Acceleration lanes may be used; however, they make it more difficult for pedestrians, especially those with visual impairments, to cross the turning roadway. Therefore, acceleration lanes are not advisable where pedestrian activity is anticipated.
6. **Drainage:** See guidance provided under *Urban Design, Drainage*. When pedestrian facilities are not initially installed, consideration should be given to determine where crosswalks may be installed in the future to avoid conflicts with inlet locations.
7. **Lighting:** See guidance provided under *Urban Design, Lighting*.
8. **Apparatus and Pole Placement:** See guidance provided under *Urban Design, Apparatus and Pole Placement*. When pedestrian facilities are not initially installed, care should be taken when placing ITS equipment, apparatuses, and signal and utility poles to avoid the anticipated or planned location of future pedestrian walkways and landing areas.
9. **Crosswalk Location:** See guidance provided under *Urban Design, Crosswalk Location*. In the event that an acceleration lane is present, consider placing the crosswalk at the upstream end of the turning roadway, placing pedestrians in the line of sight for drivers as they decelerate to make the turn. At the downstream end, on approach to an acceleration lane, drivers will be accelerating out of the turn and more likely focusing on cross-street traffic.
10. **Crosswalk Orientation:** See guidance provided under *Urban Design, Crosswalk Orientation*.
11. **Crosswalk Markings:** See guidance provided under *Urban Design, Crosswalk Markings*.
12. **Signage:** See guidance provided under *Urban Design, Signage*.
13. **Bicycle Lane:** See guidance provided under *Urban Design, Bike Lane*.

Rural Design

If pedestrian activity is expected, see the Guidelines for Suburban Design section.

If pedestrians are not an issue, the following guidelines may be used:

1. **Angle of Entry:** In rural areas, the angle of entry between the slip lane and the cross street is typically flatter than in urban areas to facilitate high-speed turns. Therefore, the channelizing island should be constructed as an isosceles triangle.
2. **Radius:** For guidance on radius design for different vehicle classes, see *Minimum Designs for Truck and Bus Turns* in Chapter 7. A design speed that is appropriate for the turning movement should be used to determine the combination of radius and superelevation for the right turn.
3. **Channelizing Island:** The channelizing island may be flush with the pavement or depressed.

   Careful consideration should be made in rural areas for the use of curbed islands, particularly along high-speed facilities and at isolated intersections. If curbs are installed, they should be sloped and offset from the traveled way, and islands should be made clearly visible to motorists. See Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets* for design guidance on curb offset and tapering (Figure D-1). Details on island approach treatment and delineation are presented in Chapter 3 of the *TMUTCD* and Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*.
4. **Deceleration Lane:** See Chapter 2, *Number, Location, and Spacing of Access Connections* of the TxDOT *Access Management Manual* for volume thresholds for installing deceleration lanes. See Chapter 3, *Multi-Lane Rural Highways, Right-Turn Deceleration Lane* in this manual for design guidance for deceleration lanes.
5. **Acceleration Lane:** See Chapter 2, *Number, Location, and Spacing of Access Connections* of the TxDOT *Access Management Manual* for volume thresholds for installing acceleration lanes. These lanes provide a benefit when right-turn volumes are especially high and/or the speed differential between turning vehicles and vehicles on the cross street is large. Acceleration lanes provide benefits to motorist by allowing them to reach a higher speed prior to merging but they may increase sideswipe crashes. Acceleration lanes have been found to be preferred by elderly drivers at high-speed intersection locations. See Chapter 3, *Two-Lane Rural Highways, Speed Change Lanes, Right Turn Acceleration Lanes* and *Multi-Lane Rural Highways, Turn Lanes, Acceleration Lanes* of this manual for design recommendations for acceleration lanes for two-lane and multi-lane rural highways, respectively.
6. **Lighting:** Intersections with channelization should be illuminated.

Section 3 — Retrofitting Treatments

Common issues encountered at right-turn slip lanes include the absence of adequate refuge in the channelizing island for crossing pedestrians, failure of motorists to stop and yield to crossing pedestrians, pedestrian noncompliance with the crosswalk location, high-speed turns jeopardizing pedestrian safety, low visibility of crossing pedestrians, and excessive head turning required to observe oncoming traffic. These observations should be supported by collision diagrams and/or crash analyses of the intersection. Potential retrofitting treatments designed to mitigate these issues are presented below.

Providing Proper Refuge for Pedestrians

Where the channelizing island along a right-turn slip lane is painted and does not provide adequate refuge for crossing pedestrians, consideration should be given to installing a raised island. Raised pedestrian islands reduce the crossing distance for pedestrians by allowing pedestrians to cross the through lanes and turning roadway separately while taking refuge on the island between. The reduction in crossing distance may also improve signal timing.

At intersections where there is a raised channelizing island, but it is not large enough to provide refuge for pedestrians, the island should be expanded to establish an adequate landing area for pedestrians and comply with ADA regulations. The minimum recommended size of the channelizing island is 300 ft^2 for intersections in all area types; the specific site conditions will dictate the final size.

Stopping and Yielding to Crossing Pedestrians

At intersections where there is a concern that motorists are failing to stop and yield to crossing pedestrians, several treatments to improve compliance can be considered. These treatments are intended to achieve one or both of the following: improve the visibility of the crosswalk and/or decrease the speed of right-turning motorists. When visibility is a concern and the crossing is currently marked, consideration may be given to upgrading the markings to a “High-Visibility Longitudinal Crosswalk” pattern. An advanced warning sign (W11-2) and stop bar may also be installed (see Figure D-3). Upgrade safety lighting as needed.

In areas where pedestrian activity is moderate to high, raised crosswalks may be installed to slow turning motorists and improve their likelihood of stopping and yielding to crossing pedestrians. However, raised crosswalks are not recommended along high-speed facilities. Signs with pedestrian-actuated flashing beacons may be installed to provide an advanced warning to approaching motorists of the need to comply with the crossing location. Caution must be taken when installing beacons where pedestrian activity is minimal and the infrequent activation of these beacons may violate driver expectations.

NCHRP *Report 562* provides additional guidance concerning different types of crossing treatments based on observed conditions, including thresholds for pedestrian and vehicle volumes and roadway speed.

Crosswalk Location

Crosswalks should be placed at a minimum of 20-ft offset from intersecting roadway, perpendicular to the direction of traffic as shown in Figures D-4 and D-5. Being the shortest path, this treatment is likely to increase compliance. Signs may be used to direct pedestrians to the location where they are expected to cross. The R9-2 sign (Figure D-6) is a regulatory sign for crossing pedestrians.

Figure D-6. R9-2 Sign (TMUTCD)

Reducing Speeds in the Channelized Roadway

High-speed turns are generally promoted by wide, sweeping turning roadways and the presence of acceleration lanes downstream of the right-turn slip lane. When applicable, consideration should be given to striping turning roadways to delineate the path for passenger vehicles and promote a sharper entry angle with the cross street. Also, the presence of a deceleration lane upstream of the turning roadway provides an area for approaching vehicles to decrease speed before making the turn while separated from through traffic. Consideration for removing the acceleration lane where their presence is not necessary (mainly along urban and suburban streets) may be appropriate as they promote high-speed turns and may cause inconsistent driver behavior (e.g., some drivers may stop or slow to look for oncoming traffic before they proceed, while others continue at pace into the acceleration lane and look for a gap closer to the downstream merge location). See Figure D-4 for the recommended design configuration of a right-turn slip lane with a deceleration lane, including pavement markings.

Enhancing Visibility of Crossing Pedestrians

At locations where the visibility of pedestrians is low, warning signs may be installed in advance of the crosswalk to alert motorists of the presence of a crosswalk ahead. Consideration should be given to upgrading markings to a “High-Visibility Longitudinal Crosswalk” pattern to enhance the visibility of the crossing location. Provision of a deceleration lane upstream of the turning roadway better accommodates a decrease in speed by approaching motorists, which provides them more time to spot crossing pedestrians. Reconstructing the turning roadway and channelizing island to incorporate a more pedestrian-friendly design may be an option as part of intersection improvements. If intersection lighting is absent or insufficient, addition or enhancement of lighting to illuminate the crossing and surrounding area may be appropriate. Other potential treatments include rectangular rapid flashing beacons, or other pedestrian-actuated traffic control devices that alert motorists to the presence of the crossing location only when pedestrians are present. This not only improves the safety conditions at these intersections, but also may reduce the impact to motorists’ mobility.

Reducing Head Turning to Spot Oncoming Traffic

At intersections where motorists are required to turn their heads excessively to observe oncoming traffic, consideration should be given to reconfiguring the channelizing island and turning roadway such that the angle of entry is closer to 70 degrees. As a result, navigating the turning roadway does not require as much physical effort to observe cross street traffic. This may involve reconstructing the channelizing island/outside curb radius or restriping the island area and turning path.

Slip Lane Removal

If none of the available right-turn slip lane treatments will address existing safety problems at the turning roadway, and pedestrian activity is very high, consideration may be made to close the slip lane and transform the area into a pedestrian-friendly corner with street furniture, benches, and landscaping. A shared through-right-turn lane would replace the slip lane to accommodate the right turning movement. However, this option should be carefully considered as the removal of the slip lane may eliminate a number of benefits, including the reduction of vehicular delays and rear-end crashes.

Section 4 — References

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8. Pedestrian Facilities Curb Ramps Ped 12-A, Texas Department of Transportation, 2012.
9. Pedestrian Refuge Island, Metropolitan Transportation Commission.
10. Texas Manual on Uniform Traffic Control Devices, Texas Department of Transportation, 2011.
11. NCHRP 3-89: Design Guidance for Channelized Right-Turn Lanes, National Cooperative Highway Research Program, Transportation Research Board.

Appendix E — Alternative Intersections and Interchanges

Contents:

Section 1 — Overview

Introduction: Alternative Intersections and Interchanges

In recent years, the number of alternative intersections and interchanges has increased substantially in the United States and in Texas. As these numbers have increased, the body of data documenting the efficacy and appropriate applications of these intersections has also increased. Some of the advantages of these types of intersections/interchanges include the following:

- An improvement in traffic flow by eliminating, relocating or modifying conflict points
- Improved signal phasing and operations
- A general decrease in crashes, and particularly a reduction in more severe crashes
- A decrease in congestion and a reduction in traffic bottlenecks
- Improvement of intersection delay, Level of Service (LOS), travel time, and vehicle throughput
- The increased ability to maintain existing bridge structures
- A possible reduction in the amount of ROW required for new projects

There are many tools currently available to conduct a preliminary (Stage 1) assessment to determine if a particular alternative is viable and preferable. The *FHWA Alternative Intersections/ Interchanges Informational Report* is a report that provides guidance on proper selection, and the *FHWA Capacity Analysis for Planning of Junctions (CAP-X)* is an Excel based program that can be used to evaluate selected types of innovative junction designs (eight intersections, five interchanges, three roundabouts and two mini-roundabouts) using given peak flow volumes. Additionally, the *FHWA Safety Performance Intersection Control Evaluations (SPICE)*, an Excel based program uses safety performance functions (SPFs) in Part C of the *Highway Safety Manual (HSM)* to evaluate and compare the safety aspects of at-grade intersection alternatives. The SPICE tool allows the selection of default safety performance functions (SPFs) and high quality crash modification factors (CMFs) from Part D of the HSM and CMF Clearinghouse to predict crash frequency and severity of intersection control strategies.

Stage 2 analysis would include a more detailed analysis of the preferred alternatives from Stage 1. These could include use of the Highway Safety Manual (HSM) procedures and associated programs such as Interactive Highway Safety Design Model (IHSDM), and Safety Analyst for safety analysis; Highway Capacity Manual (HCM) based tools such as Synchro, Sidra, and microsimulation modeling tools such as Vissim for traffic operational analysis; and cost-benefit analysis models.

The Design and Traffic Safety Divisions will be developing suggested procedures and protocols that incorporate Intersection Control Evaluation (ICE) which is a data-driven, performance-based framework and approach to objectively screen intersection alternatives and identify an optimal geometric and control solution for an intersection.

Early in the selection process of an alternative intersection, public involvement should be conducted to educate and allow feedback from the stakeholder and local community concerning the benefits and proper function of the chosen alternative, including vehicular, pedestrian, and bicyclist accommodations.

Pedestrian Considerations for Alternative Intersections

While pedestrian considerations for alternative intersections require a closer examination, general considerations for non-standard roadway design treatments include:

- Ensuring sight distance for pedestrians and motorists and minimizing obstructions, this is particularly important as vehicles and pedestrians are often approaching one another at non-right angles;
- Providing wayfinding for navigation;
- Safe and comfortable traversal of intersection;
- Reducing the turning geometry in slip lanes to decrease speed;
- Minimizing the number of crossing stages; and
- Reducing the number of multiple conflict-point locations for crossings.

For more information on non-standard roadway treatments, see NCHRP Report 948: *Guide for Pedestrian and Bicycle Safety at (Alternative) Intersections and Interchanges*.

Section 2 — Roundabouts

Overview

A roundabout is a form of a circular intersection in which traffic travels counterclockwise around a central island and entering traffic yields to the circulating traffic. Roundabouts have been demonstrated to significantly reduce the number of severe crashes at intersections, improve Level of Service (LOS), and increase capacity. TxDOT has adopted *NCHRP Report 672 (Roundabouts an Informational Guide, 2nd Edition)* as the primary source for roundabout design guidelines. The information contained in this Appendix is considered a companion guide to *NCHRP Report 672*, and is intended to document TxDOT's suggested approach to roundabout design.

Planning

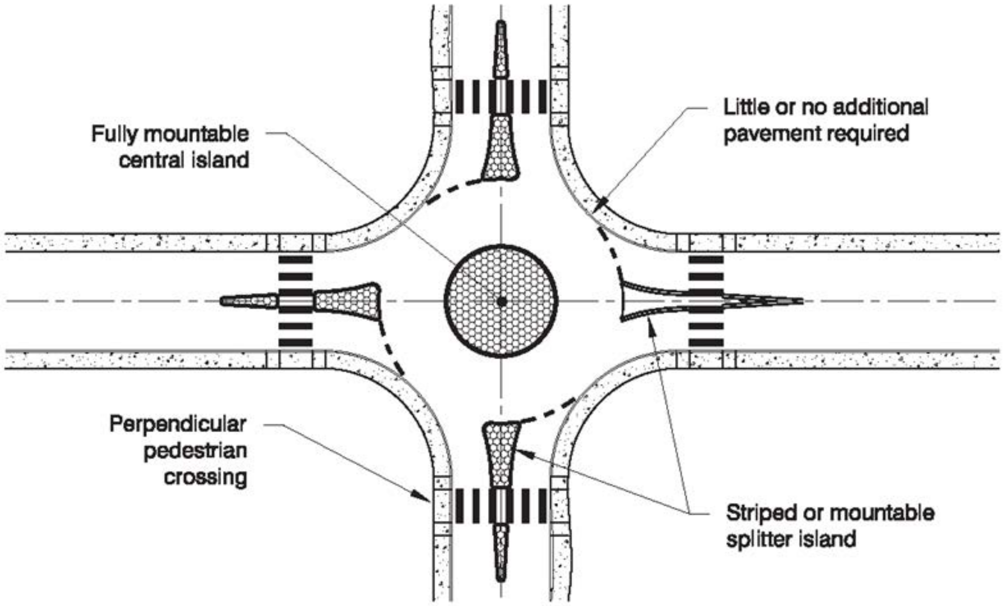
During the planning stage many considerations are needed to determine the applicability of a roundabout, including ROW, utilities, access management, operations of adjacent intersections, safety impacts, existing and predicted future traffic volume, public education, and public outreach. Locations that meet or nearly meet a signal warrant(s) should be given consideration for roundabout installation. Intersections that are, or proposed to be, all-way stop control may also be good candidates for a roundabout. Figures E-1 through E-4 show the design characteristics and features of the three roundabout categories: Mini-roundabout, Single Lane Roundabout, and Multilane Roundabout.

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Desirable maximum entry design speed	15 to 20 mph (25 to 30 km/h)	20 to 25 mph (30 to 40 km/h)	25 to 30 mph (40 to 50 km/h)
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	45 to 90 ft (13 to 27 m)	90 to 180 ft (27 to 55 m)	150 to 300 ft (46 to 91 m)
Central island treatment	Fully traversable	Raised (may have traversable apron)	Raised (may have traversable apron)
Typical daily service volumes on 4-leg roundabout below which may be expected to operate without requiring a detailed capacity analysis (veh/day)*	Up to approximately 15,000	Up to approximately 25,000	Up to approximately 45,000 for two-lane roundabout

*Operational analysis needed to verify upper limit for specific applications or for roundabouts with more than two lanes or four legs.

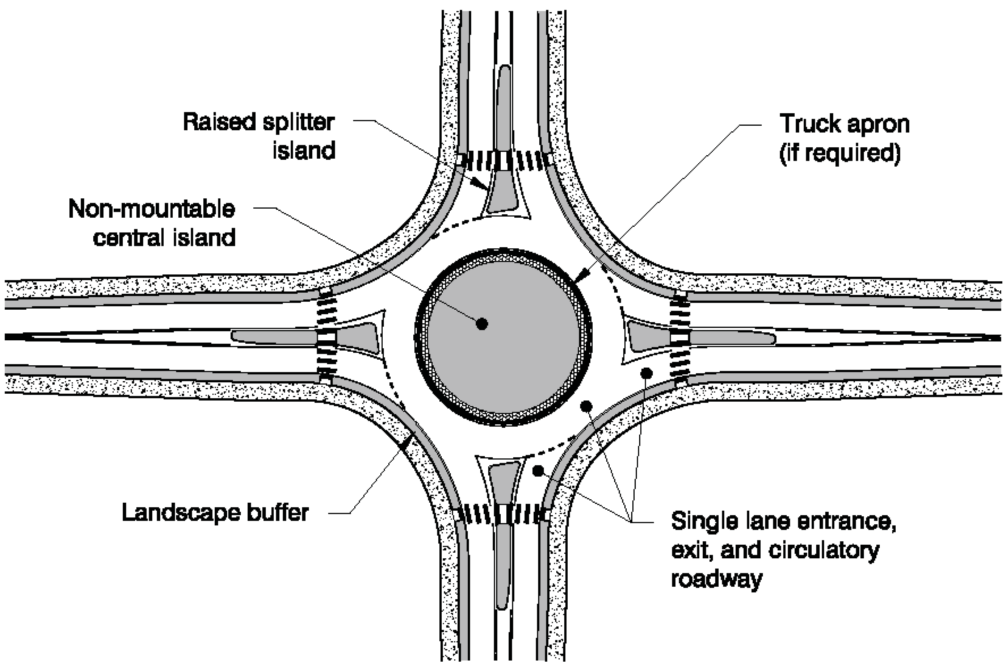
Source: NCHRP Report 672 - Exhibit 1-9

Figure E-1. Design Characteristics of Three Roundabout Categories



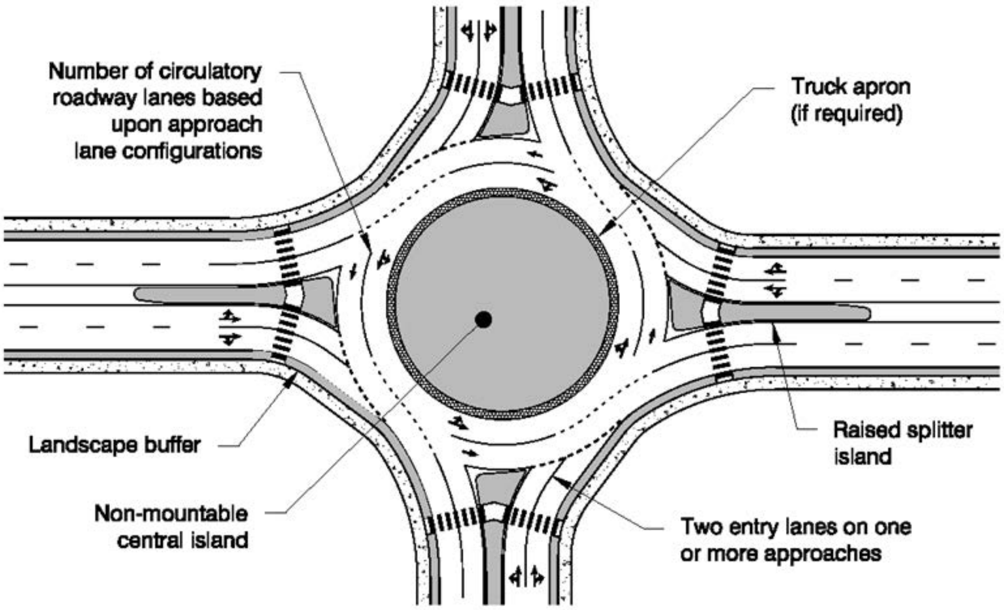
Source: NCHRP Report 672 - Exhibit 1-10

Figure E-2. Features of a Typical Mini-Roundabout



Source: NCHRP Report 672 - Exhibit 1-12

Figure E-3. Features of a Typical Single-lane Roundabout



Source: NCHRP Report 672 - Exhibit 1-14

Figure E-4. Features of a Typical Two-lane Roundabout

The preference in Texas is to utilize a single lane roundabout as long as possible to avoid unnecessarily overbuilding the roundabout. Experience shows that most roundabouts that are redesigned are actually reduced in size (e.g., 2 to 1 lane). In some instances, the reduction of lanes was done to diminish minor crash patterns[5]. If a single lane roundabout is found to be adequate for up to 10 years after the opening year, a single lane roundabout should be constructed. If a multilane roundabout is required before the design year (20 years after opening year), the single lane roundabout should be constructed having the footprint of a multilane roundabout and be designed to be easily retrofitted to a multilane roundabout when needed. For multilane roundabout designs, additional independent reviews are recommended by designers with expertise in roundabout design.

Geometric Design

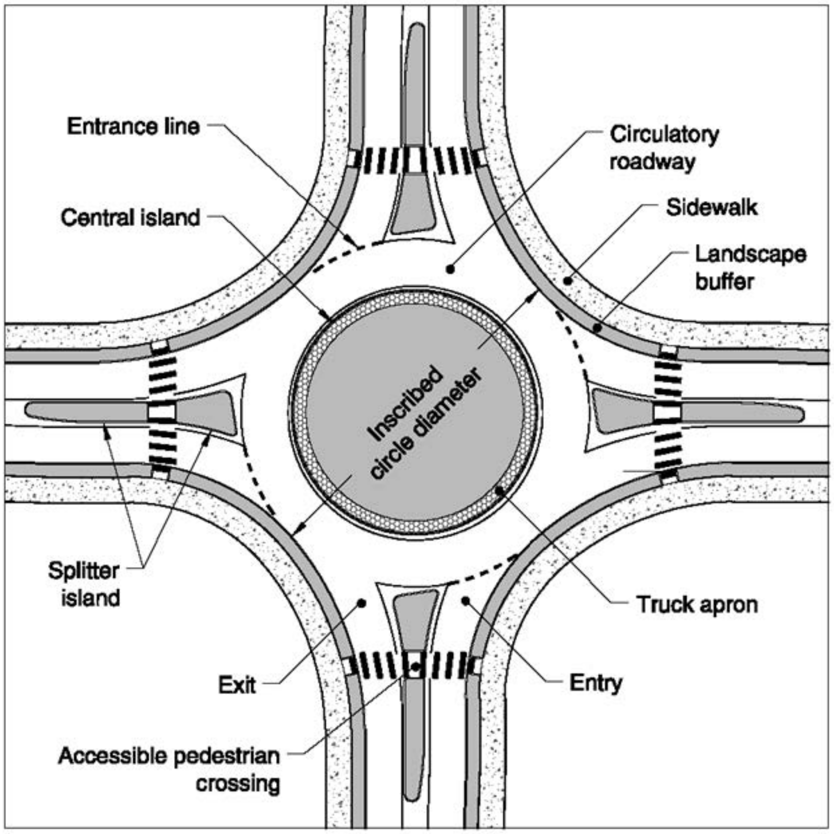
The primary goals of any roundabout design are to provide:

- Slow entry and consistent speeds throughout the roundabout by using deflection.
  - Deflection can be achieved by providing a curvilinear path upon entry into the roundabout to help control entry speeds. This will force drivers to enter at slow speeds and promote consistency in speeds between traffic entering, circulating, and exiting the roundabout.
- The appropriate number of lanes and lane assignment to achieve capacity, lane volume balance, and lane continuity.
- Smooth channelization that result in vehicles naturally using the intended lanes.

- Adequate accommodation for the design vehicles.
- Safe accommodations for pedestrians and bicyclists.
- No more than the minimum intersection sight distance for driver recognition of the intersection and conflicting users.

The two most relevant aspects of sight distance for roundabouts are stopping sight distance and intersection sight distance. Stopping sight distance must be provided at every point within a roundabout, and on each entering and exiting approach. At roundabouts, the only locations requiring evaluation of intersection sight distances are the entries. A combined sight distance diagram should be overlaid onto a single drawing to illustrate the clear vision areas for the intersection. Chapter 6 of *NCHRP Report 672* provides specific guidance with respect to sight distance determination and application.

The design vehicle is the largest vehicle likely to use the alternative intersection; this is typically a WB-67 vehicle in Texas (a smaller design vehicle may be used for urban/local or suburban classifications). The anticipated truck movements through the roundabout should be determined in AutoTURN, AutoTrack, or a similar tool used to determine vehicle encroachment within the roundabout. Currently there is research underway to update the vehicle turning movement templates since they are decades old and tend to be overly conservative. If a significant number of large trucks is anticipated through an intersection, it is advisable to find an area where the wheel paths of the respective design vehicle could be physically tested and documented. The largest truck movements may be accommodated by using a truck apron (with a roll down curb) within the central island. The apron provides additional paved surface for the design vehicle while keeping the actual circulatory roadway width narrow enough to maintain speed control for smaller passenger cars. The basic geometric elements of a roundabout and typical inscribed circle diameters are shown in Figures E-5 and E-6:



Source: NCHRP Report 672 - Exhibit 6-2

Figure E-5. Basic Geometric Elements of a Roundabout

Roundabout Configuration	Typical Design Vehicle	Common Inscribed Circle Diameter Range*	
Mini-Roundabout	SU-30 (SU-9)	45 to 90 ft	(14 to 27 m)
Single-Lane Roundabout	B-40 (B-12)	90 to 150 ft	(27 to 46 m)
	WB-50 (WB-15)	105 to 150 ft	(32 to 46 m)
	WB-67 (WB-20)	130 to 180 ft	(40 to 55 m)
Multilane Roundabout (2 lanes)	WB-50 (WB-15)	150 to 220 ft	(46 to 67 m)
	WB-67 (WB-20)	165 to 220 ft	(50 to 67 m)
Multilane Roundabout (3 lanes)	WB-50 (WB-15)	200 to 250 ft	(61 to 76 m)
	WB-67 (WB-20)	220 to 300 ft	(67 to 91 m)

* Assumes 90° angles between entries and no more than four legs. List of possible design vehicles is not all-inclusive.

Source: NCHRP Report 672 - Exhibit 6-9

Figure E-6. Typical Inscribed Circle Diameter Ranges

Entry Width

The entry width is determined by the design vehicle entering the roundabout. For single-lane entrances a typical range for entry width would be from 16 to 20-ft. A 17-ft entry width would be considered a good starting point. Entry widths greater than 20-ft often confuse drivers into thinking there are two lanes available when there is only one circulatory lane in the roundabout. Chapter 6 of *NCHRP Report 672* provides additional information on entry widths for roundabouts.

Circulatory Roadway Width

The circulatory width is generated from the number of lanes entering the roundabout and the turning movements of the design vehicle. For single-lane roundabouts, an 18 to 20-ft circulatory roadway width would be typical. For two-lane roundabouts, it may be necessary to accommodate turning movements for one of the following:

- Two passenger vehicles side by side assuming infrequent traffic from semi-trailers;
- A passenger vehicle and single unit truck side by side;
- Where semi-trailer traffic is greater than 10 percent of the overall traffic, a semi-trailer in conjunction with a passenger vehicle or single unit truck.

Chapter 6 of *NCHRP Report 672* provides additional information on circulatory roadway widths for roundabouts.

Entry Curve

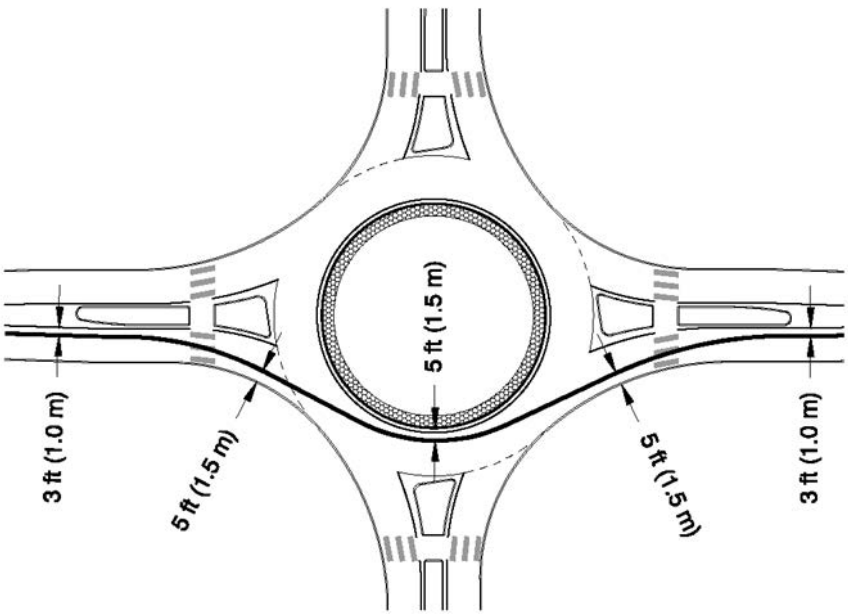
Capacity and safety are both impacted by the determination of the entry curve radius. Large entry curves may generate relatively fast entry speeds, and reductions in capacity are generally noticed with entry curves less than 50-ft. Entry curves with a radius of 50 to 100-ft are typical for an urban single-lane roundabout. Entry radii from 70 to 85-ft would be a good beginning point for determining the entry radius of a multilane roundabout. Chapter 6 of *NCHRP Report 672* provides additional information on entry curves for roundabouts.

Exit Curve

The exit curve design is controlled by conditions such as pedestrian traffic, whether it is an urban roundabout or a rural roundabout, and geometric limitations. In order to avoid congestion and crashes at the exit point in a roundabout, exit curve radii are typically greater than entry curve radii. Exit curve radii of 200 to 400-ft is recommended to promote proper tangent alignment from the circulatory to the exit. Larger trucks are more easily accommodated by larger exit curve radii.

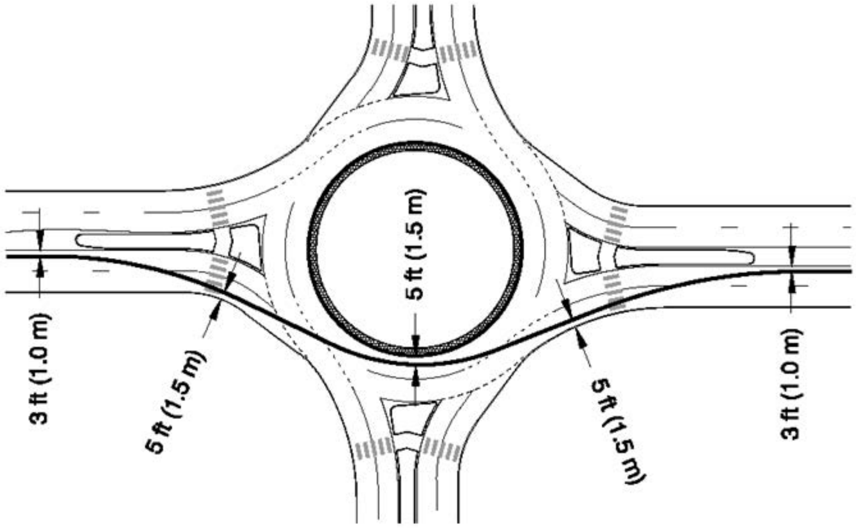
Roundabout Speeds

The most critical design objective for a roundabout is to maintain low and consistent speeds at the entry and throughout the roundabout. Horizontal curvature and reduced pavement widths influence low speed conditions in roundabouts. The maximum entering design speeds based on a theoretical fastest path are: Mini-Roundabout – 20 mph; Single Lane – 25 mph; Multilane – 25 to 30 mph. See Figure E-7 and Figure E-8 for depictions of the fastest vehicle path through a single lane-lane roundabout and a multilane roundabout, respectively. See Chapter 6 of *NCHRP Report 672* for recommended horizontal curvatures and pavement widths to produce proper speeds.



Source: NCHRP Report 672 - Exhibit 6-48

Figure E-7. Fastest Vehicle Path through Single-lane Roundabout



Source: NCHRP Report 672 - Exhibit 6-49

Figure E-8. Fastest Vehicle Path through Multilane Roundabout

See *NCHRP Report 672* for additional discussion on roundabout geometric guidance.

Vertical Geometry

In forming the vertical geometry for a roundabout, a thorough investigation of the approach roadways and central island profile is needed. The approach roadway profiles should connect to the profile for the circumference of the central island. The vertical geometry for the circumference of the central island is typically a sine curve. Examples of vertical geometry for roundabouts are shown in Figures E-9 and E-10. Note that cross slopes and/or truck apron slopes may be less than those shown in the examples to further accommodate low boy type vehicles.

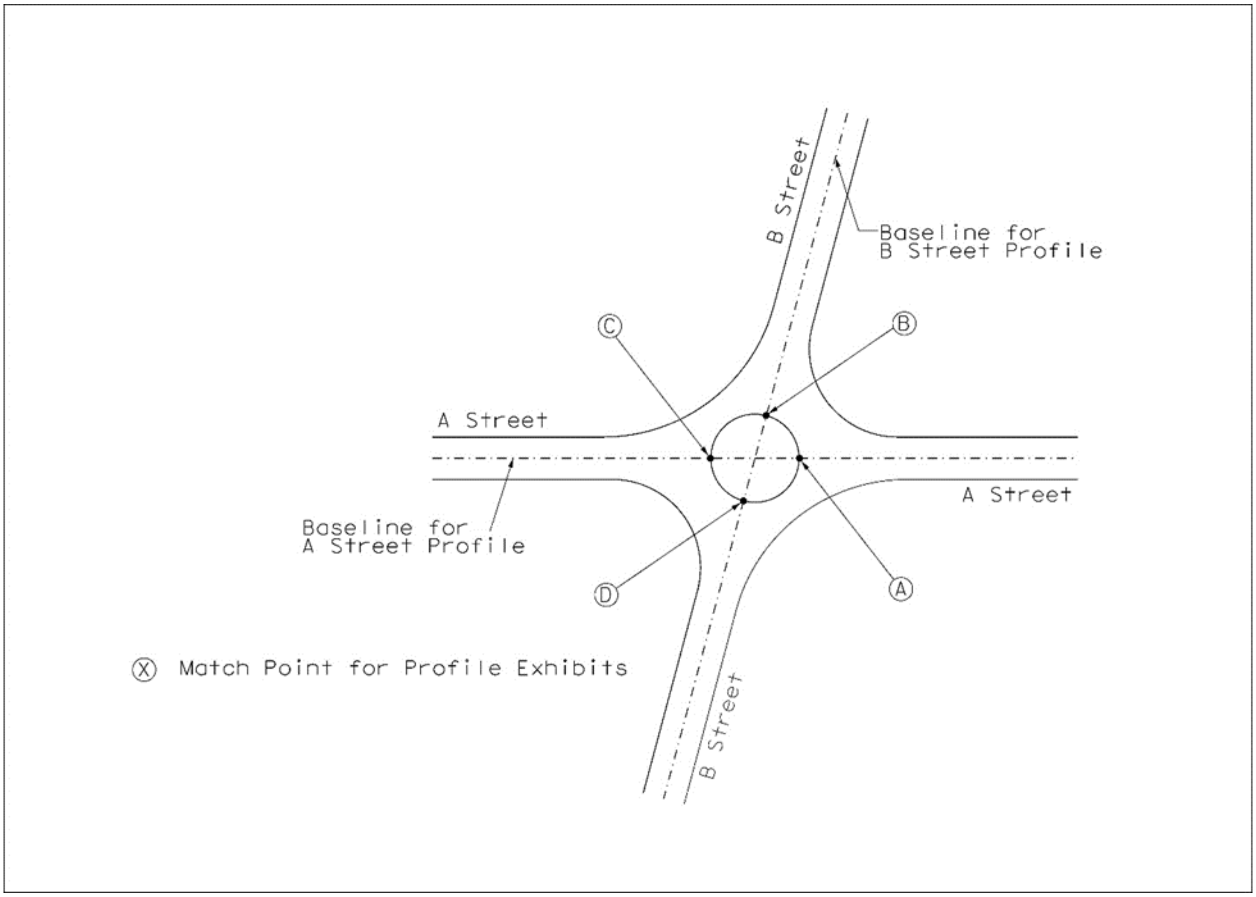


Figure E-9. Sample Central Island Profile

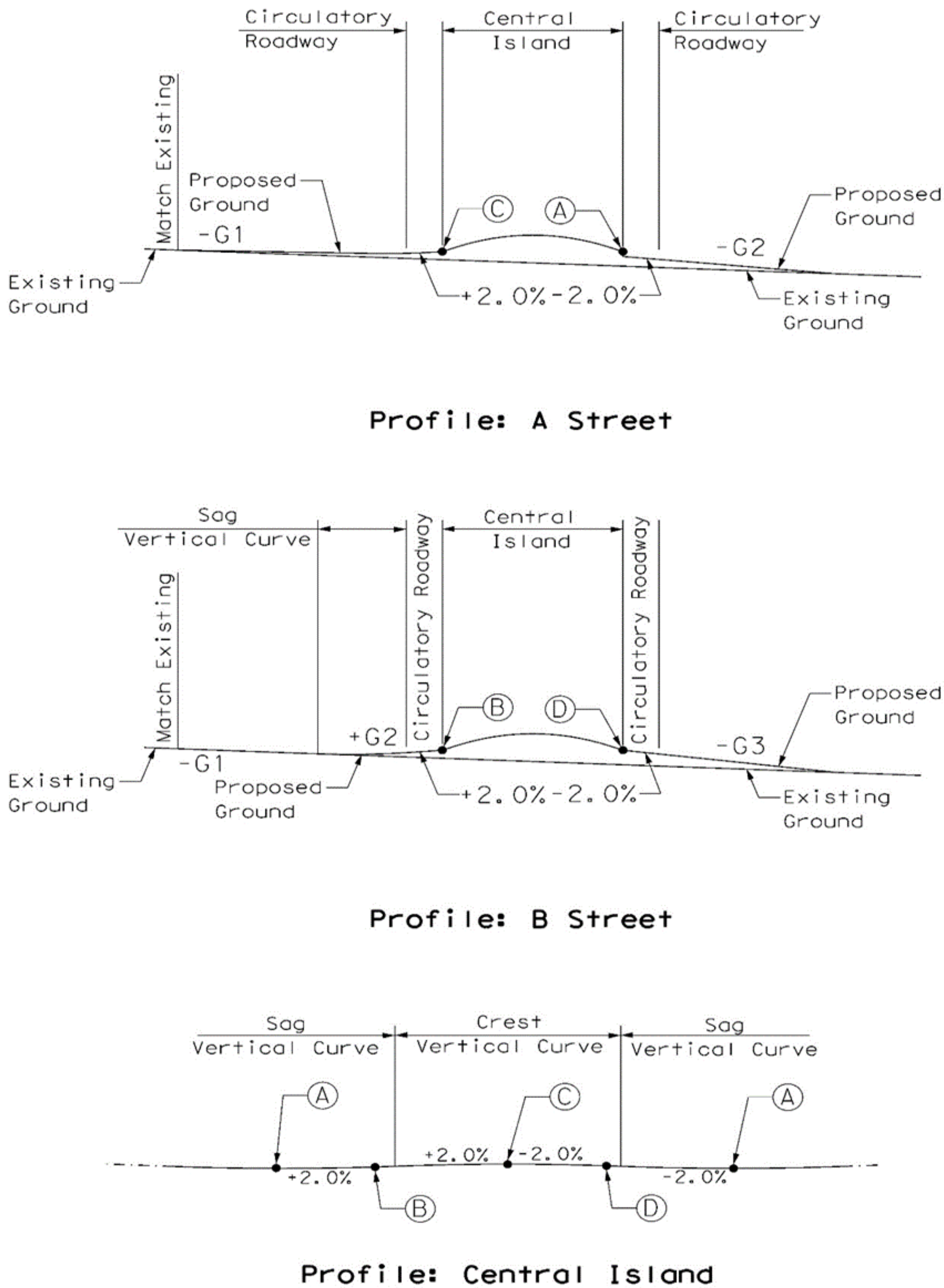


Figure E-10. Sample Central Island Profile

NOTE: All grades shown are examples.

Cross-Slope

The typical cross-slope for a roundabout is an outward 2 percent slope from the central island. Note that the cross slope may be less than 2% to further accommodate lowboy type vehicles.

Splitter Islands

AASHTO standard guidelines govern the design of the splitter island. See Figure E-11 for minimum values for nose radii and offsets. Note that larger radii than those specified in the minimums are recommended in order to avoid the breaking off of curbs.

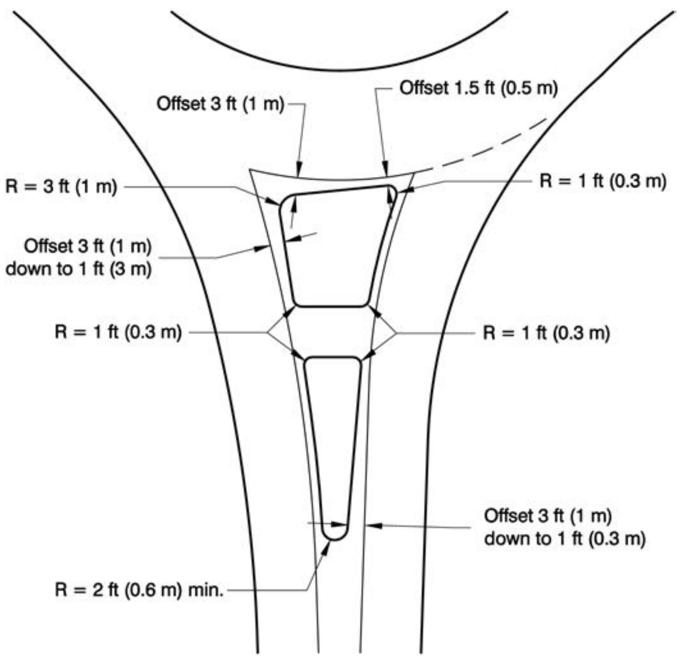
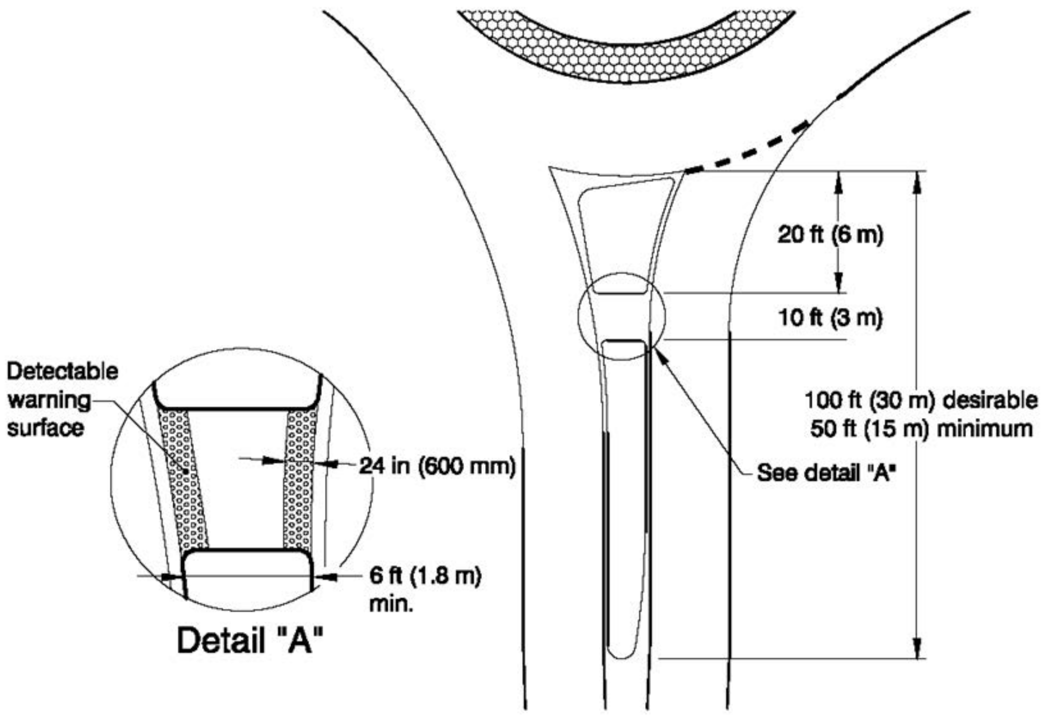


Figure E-11. Minimum Splitter Island Nose Radii and Offsets

Pedestrian Considerations

Roundabouts can provide benefit to pedestrians through reducing the number of potential conflict points with vehicles. However, roundabout entrances/exits are uncontrolled crossings for pedestrians, and designers should consider treatments in this light. Pedestrian street crossings at roundabouts can be difficult to identify for pedestrians with vision disabilities because the crossings are located to the side of the pedestrian circulation path around the street or highway.

Pedestrians should be considered and accommodated at all roundabout intersections. In some situations (such as rural intersections), pedestrian accommodations may not be necessary; however, it is recommended that splitter islands be designed to be wide enough to accommodate potential future crossings of pedestrians. By providing refuge on the splitter island, the pedestrians cross only one direction of conflicting traffic at a time. The minimum dimensions of the splitter islands are shown in Figure E-12 (Note that the 6-ft measured in Detail “A” is from back of curb to back of curb). Wherever possible, sidewalks at roundabouts should be set back from the edge of the circulatory roadway with a landscape strip. Further information for the design of general pedestrian accommodations for roundabouts is provided in NCHRP Report 672, Section 6.8.1.



Source: NCHRP Report 672 - Exhibit 6-12

Figure E-12. Minimum Splitter Island Dimensions

Considerations

Truck aprons (refer to Chapter 7 Section 3.6.3.3) can help achieve desired deflection for reducing vehicle speeds, while accommodating large vehicles in roundabouts as well as within the approach and departure lanes to the roundabouts.

Single-Lane Roundabouts

If a single-lane roundabout is located in an urban area with high expected pedestrian and vehicle traffic, consider an activated crossing beacon for pedestrians.

Multilane Roundabouts

Pedestrian facilities at multilane roundabouts should be more proactively considered and designed to help pedestrians navigate safely and comfortably. Designs with multilane exits create the greatest issues for pedestrians as they share the same multiple conflict points as a stop and yield condition midblock crossing. Drivers exiting the roundabout, though slowed by deflection of the center island and roadway edge, may not have adequate stopping distance to accommodate pedestrians entering the roadway.

Multilane roundabouts can be problematic for pedestrians with vision disabilities. Specifically, they may have difficulty detecting gaps and determining that motorists have stopped and yielded. To improve pedestrian safety, pedestrian-activated beacons, signals, or raised crosswalks may be considered at all multilane roundabout entries and exits. Detectable edging or barriers must be provided where pedestrian crossings are not intended (refer to Chapter 7 Section 3.4.4).

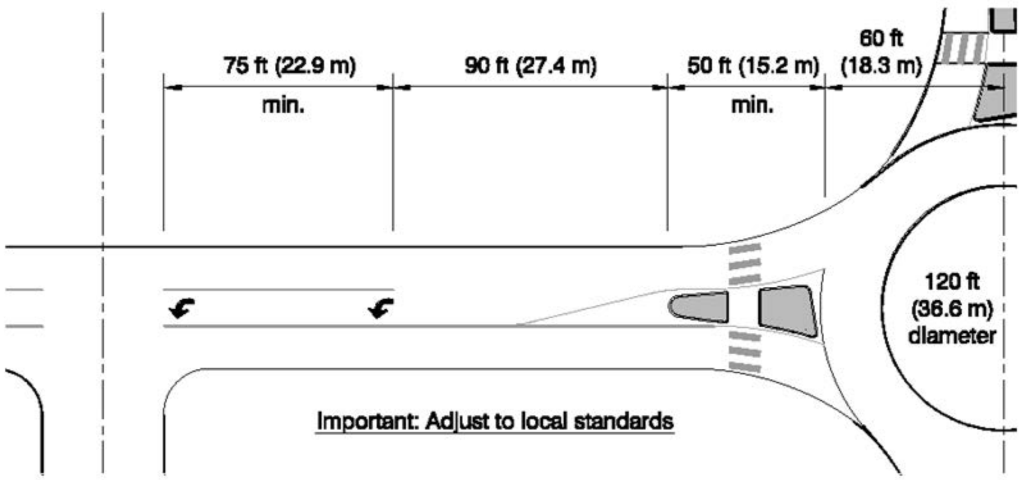
Bicyclist Considerations

Roundabouts also provide additional safety for bicyclists due to the slower vehicular speeds which result in less of a speed differential between bicyclists and motor vehicles since typical on-road bicycle speeds are between 10 and 20 mph. Bicyclists will typically take a full lane to traverse the roundabout. Less experienced bicyclists may also have the option of choosing to travel on the sidewalks provided for pedestrians and bicyclists at the roundabout. In these instances, slip ramps (which is a type of ramp for getting bicyclists from the roadway onto the sidewalk) should be considered. Chapter 6 of *NCHRP Report 672* provides additional specific guidance with respect to bicycle design at roundabouts.

Access Management

Access management is central to the proper functioning of a roundabout from an operational and safety standpoint. Avoid locating driveways with direct access to a roundabout. If unavoidable, *NCHRP Report 672* provides guidance on situations where driveways may be allowed direct access. Additionally, access points near a roundabout have restricted operations due to the channelization of the roundabout. The ability to provide full access is governed by the following factors:

- The capacity of the minor movements at the access point.
- The need to provide left turn storage downstream of the roundabout on the major street to serve the access point in order to prevent blocking the major street flow.
- The available space between the access point and the roundabout in order to allow for adequate splitter island design and left-turn pocket channelization (see Figure E-13 for an example). Note the resulting storage and taper length should be analyzed using the anticipated turning volume into the driveway to avoid queue spillback into the roundabout.
- Sight distance needs.



- *Sight distance needs.* A driver at the access point should have proper intersection sight distance and should be visible when approaching or departing the roundabout, as applicable.

Source: NCHRP Report 672 - Exhibit 6-91

Figure E-13. Typical Dimensions for Left-turn Access near Roundabouts

Reference *NCHRP Report 672* for additional guidance with respect to access management at roundabouts.

Section 3 — Diverging Diamond Interchange (DDI)

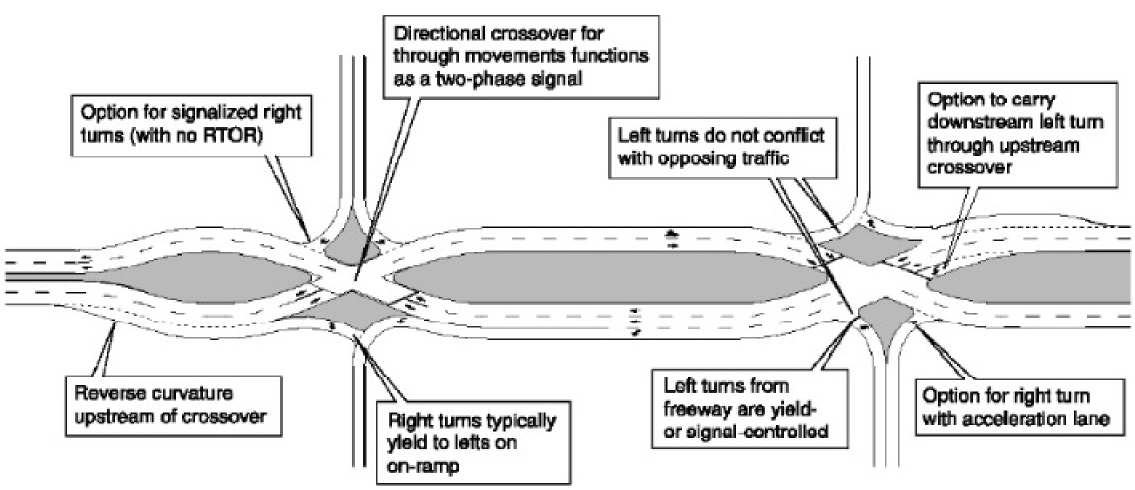
Overview

The Diverging Diamond Interchange (DDI) is an interchange form that allows the two directions of traffic on the crossroad to temporarily divide and cross to the opposite side to gain access to and from the freeway more easily. The primary difference between a DDI and a conventional interchange is the design of directional crossovers on either side of the interchange. This eliminates the need for left-turning vehicles to cross the path of approaching vehicles. By shifting cross street traffic to the left side of the street between the signalized intersections, vehicles on the crossroad making a left turn on to or off of ramps do not conflict with vehicles approaching from other directions. TxDOT has adopted the *FHWA Diverging Diamond Interchange Informational Guide*[6] as the primary source for DDI guidelines. The information contained in this Appendix is considered a companion guide and is intended to document TxDOT's suggested approach to DDI design.

Some of the documented benefits of a DDI include:

- Capacity improvements with two-phase signal configurations.
- Safety improvements due to a reduction in the number of conflict points.
- Possible lower costs due to a smaller footprint, shorter construction time, and the possibility of being able to salvage existing bridge structures.

Figure E-14 shows the design characteristics and key features of the DDI.



Source: FHWA DDI Informational Guide - Exhibit 1-1

Figure E-14. Key Characteristics of a DDI

Design Considerations

Appropriate geometrics are central in the proper functioning of a DDI. The design vehicle is typically a WB-67 vehicle in Texas. The anticipated truck movements through the DDI should be determined and AutoTURN (or a similar tool) used to determine vehicle encroachments within the DDI. There is research underway to update the current turning movement templates since they are decades old and tend to be conservative. If a significant number of large trucks is anticipated through an intersection, it is advisable to find an area where the wheel paths of the respective design vehicle could be physically tested and documented. The largest truck movements may be accommodated by using a roll down curb in turning areas. The design speed of the DDI affects the reverse curve radii through the two intersection crossovers, and should typically range from 25 to 35 mph (see RDM Chapter 2, Section 4, Table 2-5 for corresponding curve radii values). The DDI profile should be relatively flat to increase driver sight distance. A tangent section is recommended before and after the crossovers to minimize the likelihood of wrong way maneuvers into opposing lanes, and the recommended crossover angle is 40-50 degrees or greater (see Figures E-15 and E-16 respectively). Figure E-17 depicts some typical curve radii ranges for lower-speed DDIs.

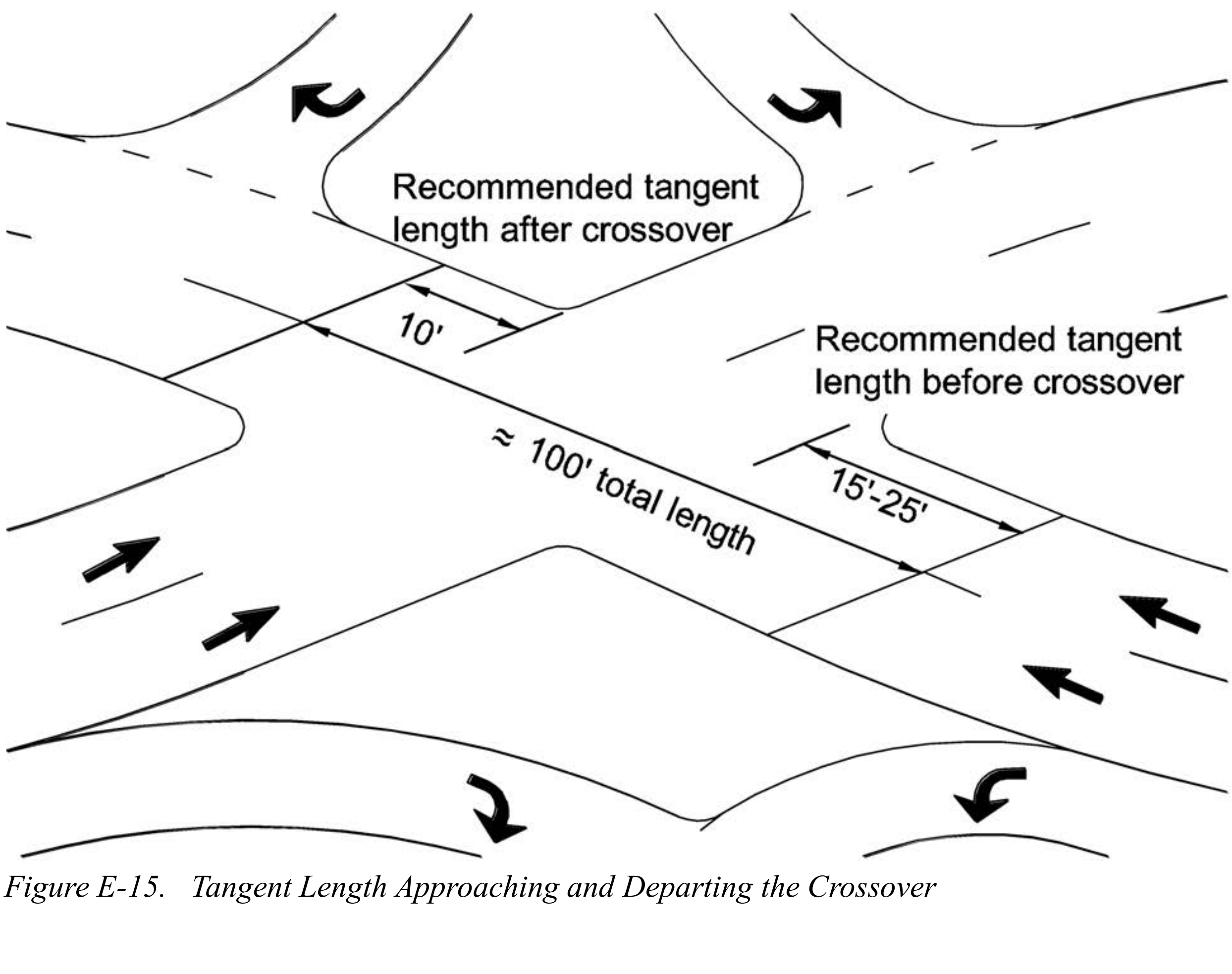
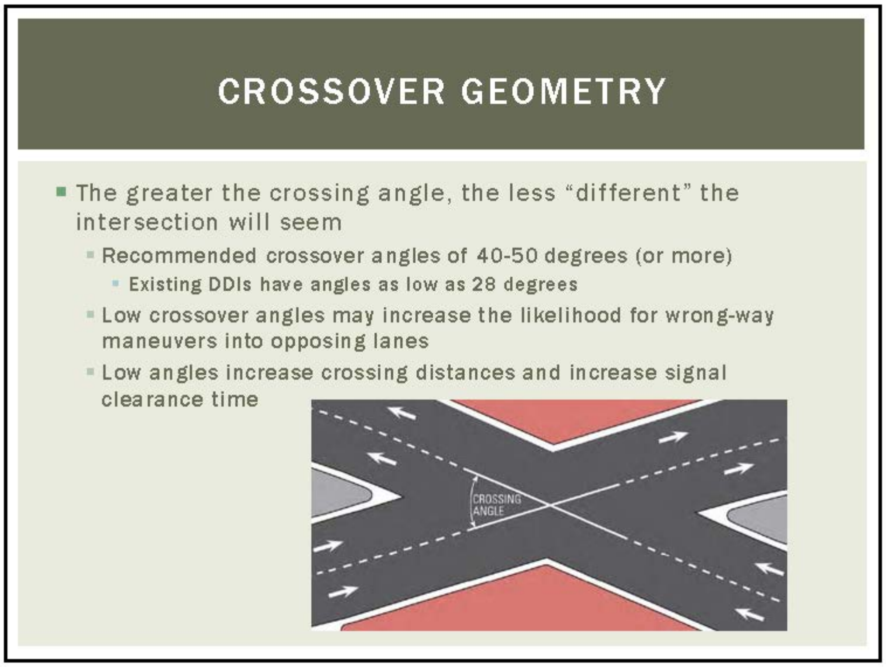
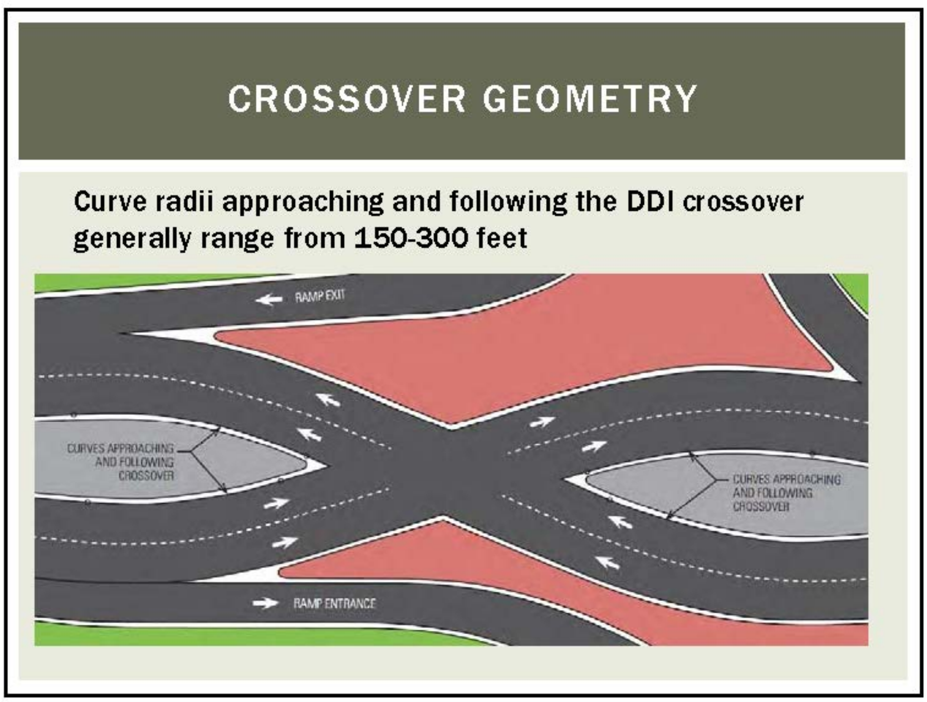


Figure E-15. Tangent Length Approaching and Departing the Crossover



Source: FHWA Intersection and Interchange Geometrics (IIG) Workshop (May 2016)

Figure E-16. Crossover Geometry (Crossing Angle)



Source: FHWA Intersection and Interchange Geometrics (IIG) Workshop (May 2016)

Figure E-17. Crossover Geometry (Curve Radii)

Sight Distance

Drivers approaching or departing an intersection must have an unobstructed view of traffic control devices and sufficient length along the cross road to safely navigate the intersection. Insufficient sight distance is a significant factor in street crashes and near collisions. As with any other intersection, DDI intersections must provide stopping sight distance (SSD) and intersection sight distance (ISD). Sight distances must be checked for these conflict areas: walls, railings, tall landscaping, or other obstructions that may limit sight distance. Intersection sight distance may also be limited by barriers or other obstacles between the crossovers.

Horizontal Alignment Alternatives

There are three alignment alternatives resulting in a minimum cross-section along the cross road regardless of whether the facility is an over-or under pass. There are two types of alignments: Symmetrical Alignment and the Shifted Alignment. Figure E-18 shows an example of symmetrical and shifted alignments. This results in distances of 600 to 750-ft between crossovers. If the distance between crossovers can be reduced it can boost traffic operations and limit the amount of right of way needed.

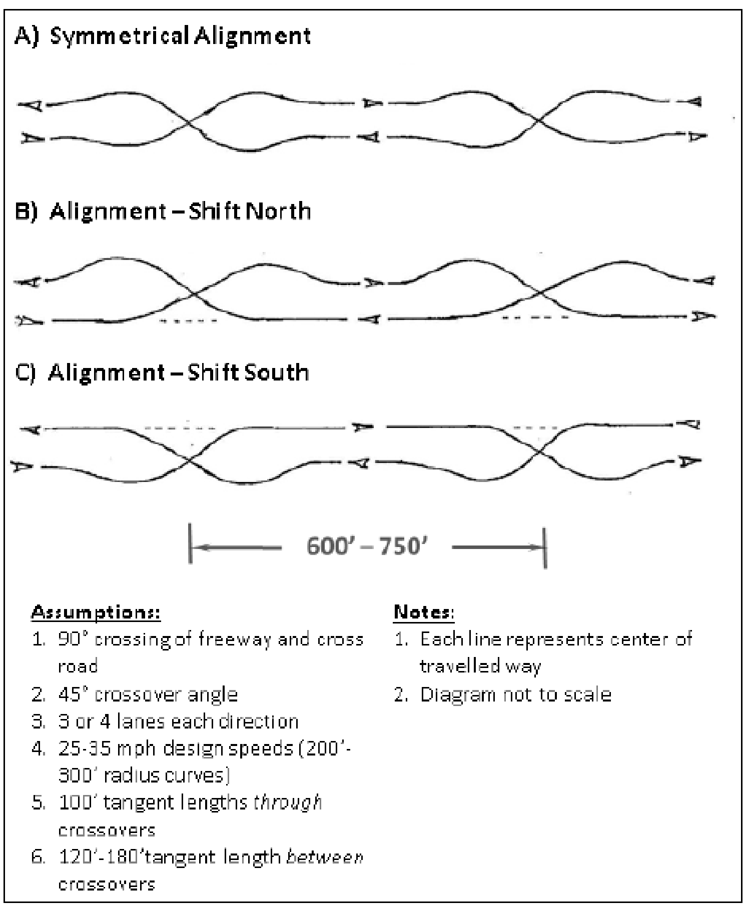


Figure E-18. Alignment Alternatives

Eliminating a few reverse curves can reduce the spacing between crossovers. Figure E-19 shows alignment alternatives where the number of reverse curves has been reduced with a resultant increase in median width. This results in distances of 400 to 500-ft between crossovers. Chapter 7 of the *FHWA Diverging Diamond Interchange Informational Guide*[6] provides additional information on horizontal alignment options.

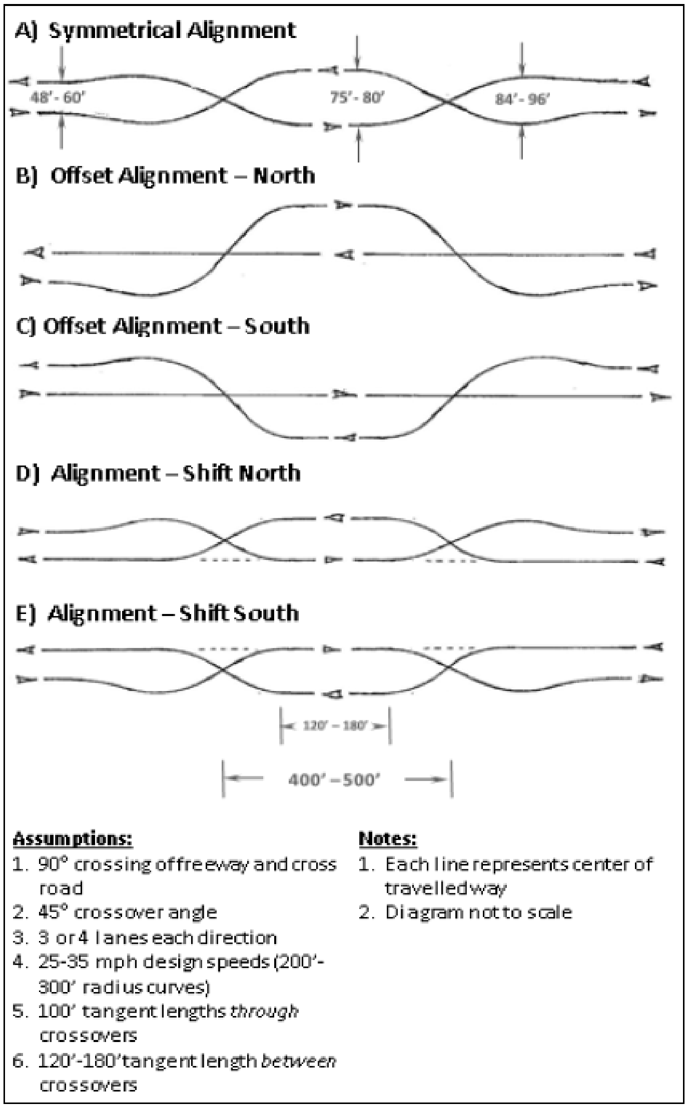


Figure E-19. Reduced Reverse Curves in Alignment Alternatives

Auxiliary Lanes

Auxiliary lanes would aid weaving traffic at a DDI interchange, smooth traffic flow, provide added capacity, and improve overall safety. Figures E-20, E-21, E-22, and E-23 show some examples of auxiliary lanes at a DDI interchange. Chapter 7 of *FHWA Diverging Diamond Interchange Informational Guide*[6] provides additional information on auxiliary lanes.

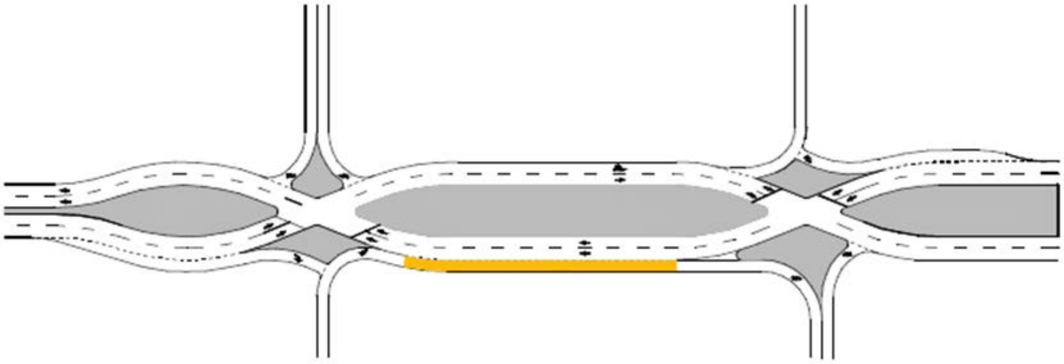


Figure E-20. Auxiliary Left Turn Lane Between Crossovers (Not Preferred)

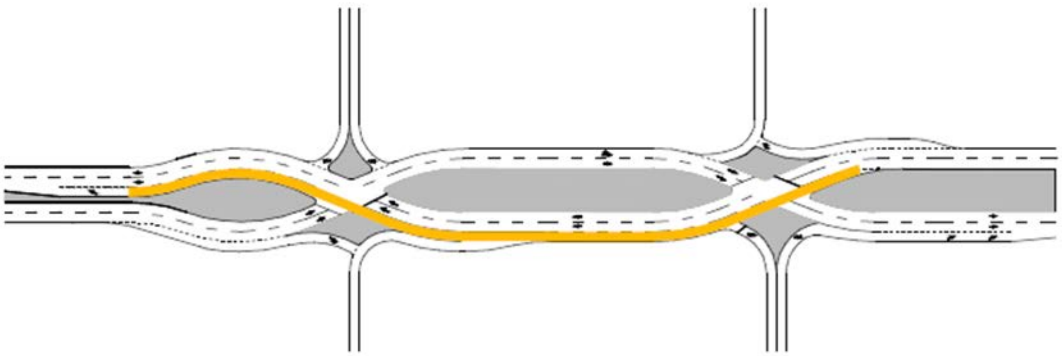


Figure E-21. Auxiliary Left Turn Lane Developed Prior to the First Crossover

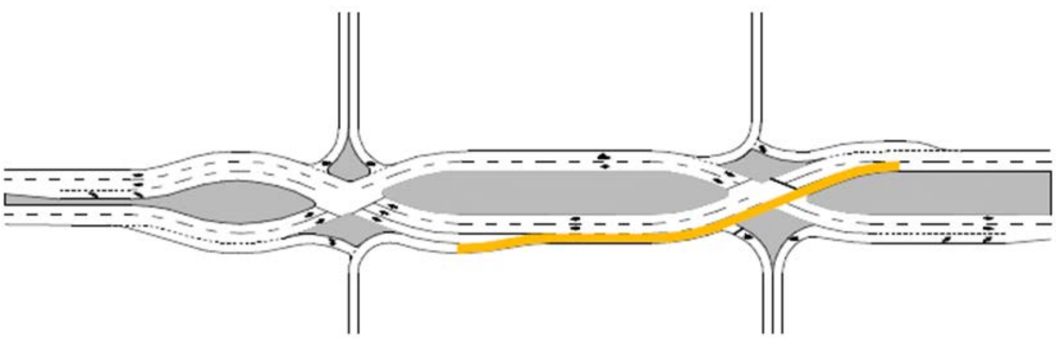


Figure E-22. Auxiliary Lane, Shared Left and Through, Developed Prior to the First Crossover

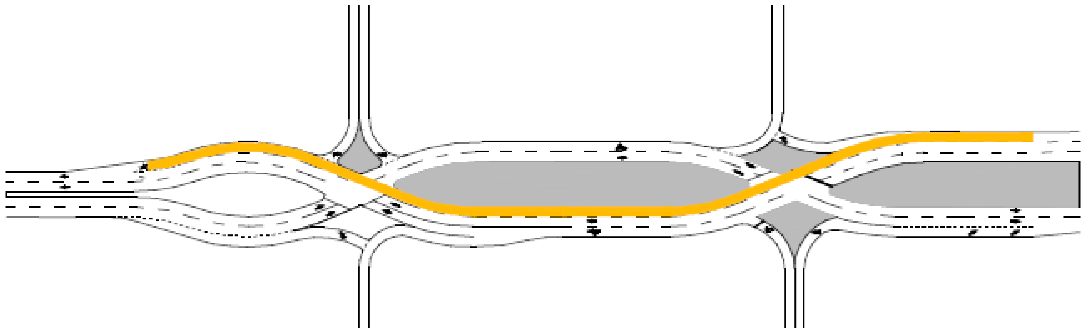
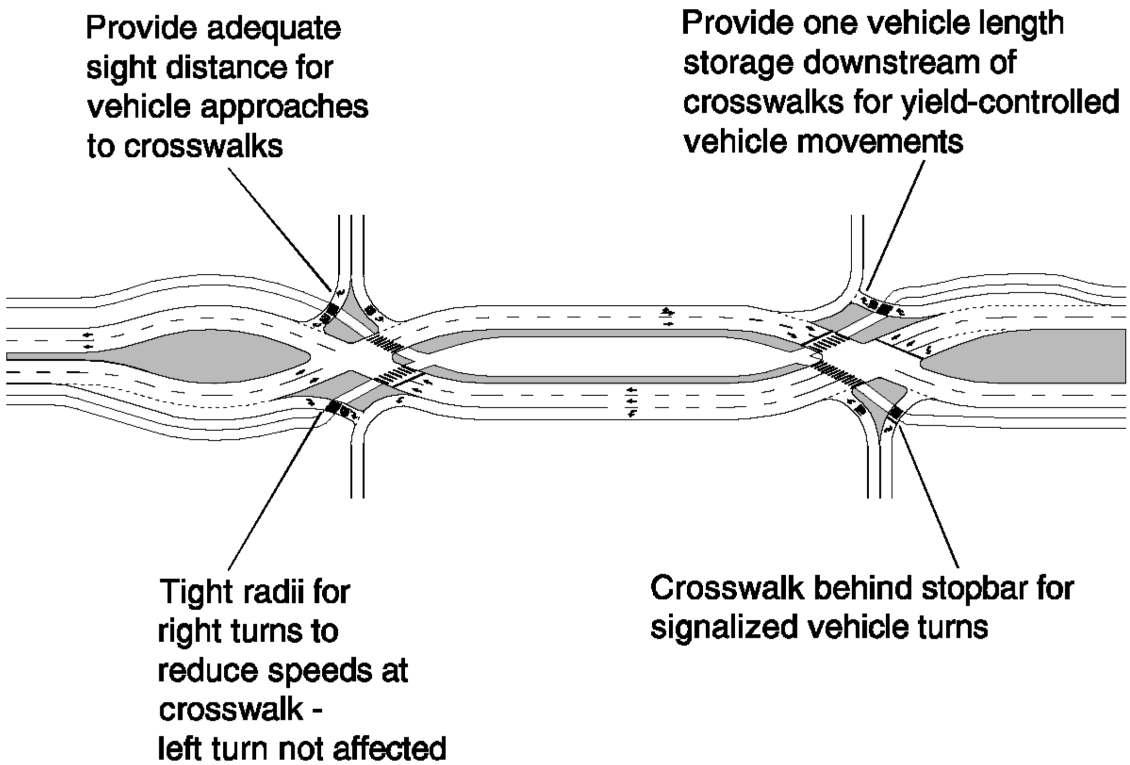


Figure E-23. Auxiliary Through Lane Developed Prior to the First Crossover

Pedestrian and Bicyclist Considerations

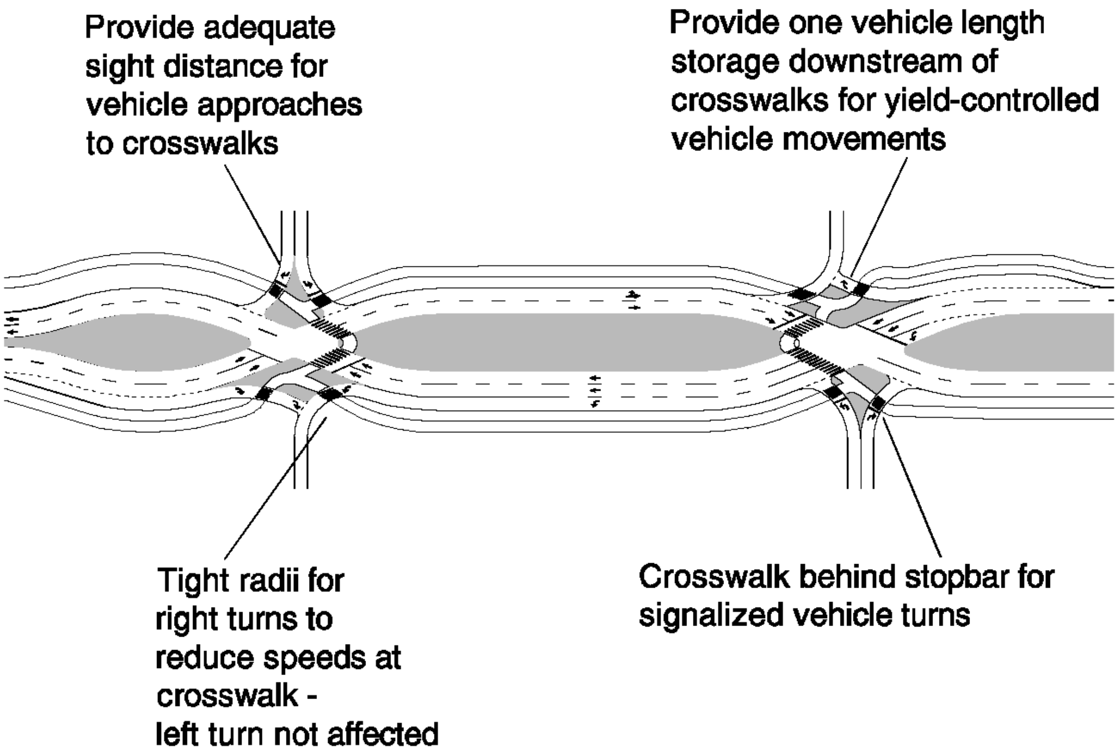
DDIs have many benefits for pedestrians which include: Allowing more crossing time per phase due to the two-phase signal operations, crossing only one direction of traffic resulting in reduced conflicts, and fewer travel lanes for a pedestrian to cross.

DDIs provide the option to direct pedestrians to either the outside of the intersection or to a center walkway. The wide area between opposing traffic allows the opportunity for a large sidewalk down the center that can be bordered by concrete barrier to allow additional protection and channelization for the pedestrians. Pedestrian facilities on the inside minimize conflicts with traffic turning left to and from the freeway, and allow the crossing of the interchange in all directions. When placing the pedestrian sidewalk along the outside some important considerations are: the location of the crosswalk with respect to the bridge structure or other sight obstructions to maintain good visibility for both the pedestrians and vehicles, and considering the turning radii to reduce speeds in the vicinity of pedestrians. Figures E-24 and E-25 depict the inside and outside pedestrian sidewalk options, respectively.



Source: FHWA DDI Informational Guide - Exhibit 3-9

Figure E-24. Pedestrian-focused DDI – Center Walkway



Source: FHWA DDI Informational Guide - Exhibit 3-10

Figure E-25. Pedestrian-focused DDI – Outside Walkway

For bicyclists, the design should focus on minimizing bicycle conflicts with motor vehicles, providing adequate lateral space between vehicles and bicycles, minimizing speed differential between bicycle and vehicles, and managing bicycle-pedestrian conflicts. The 2 primary options for bicyclists on a cross street through a DDI are:

- A marked bicycle lane through the DDI. If a separate bicycle lane is provided, the preference is to locate it to the right of the vehicular traffic. Bicycle lane widths of 5 to 7-ft. are recommended through the DDI. (See Figure E-26)
- A separated sidewalk or wider shared-use path. This would typically entail the bicyclist disembarking on the upstream end of the DDI and then proceeding through the DDI in the same area designated for pedestrians.

Ultimately, a thorough site assessment, an assessment of anticipated bicycle and pedestrian volumes, and an assessment of projected origins and destinations for pedestrians and bicyclists should be conducted to determine the preferred method of movement through the DDI.

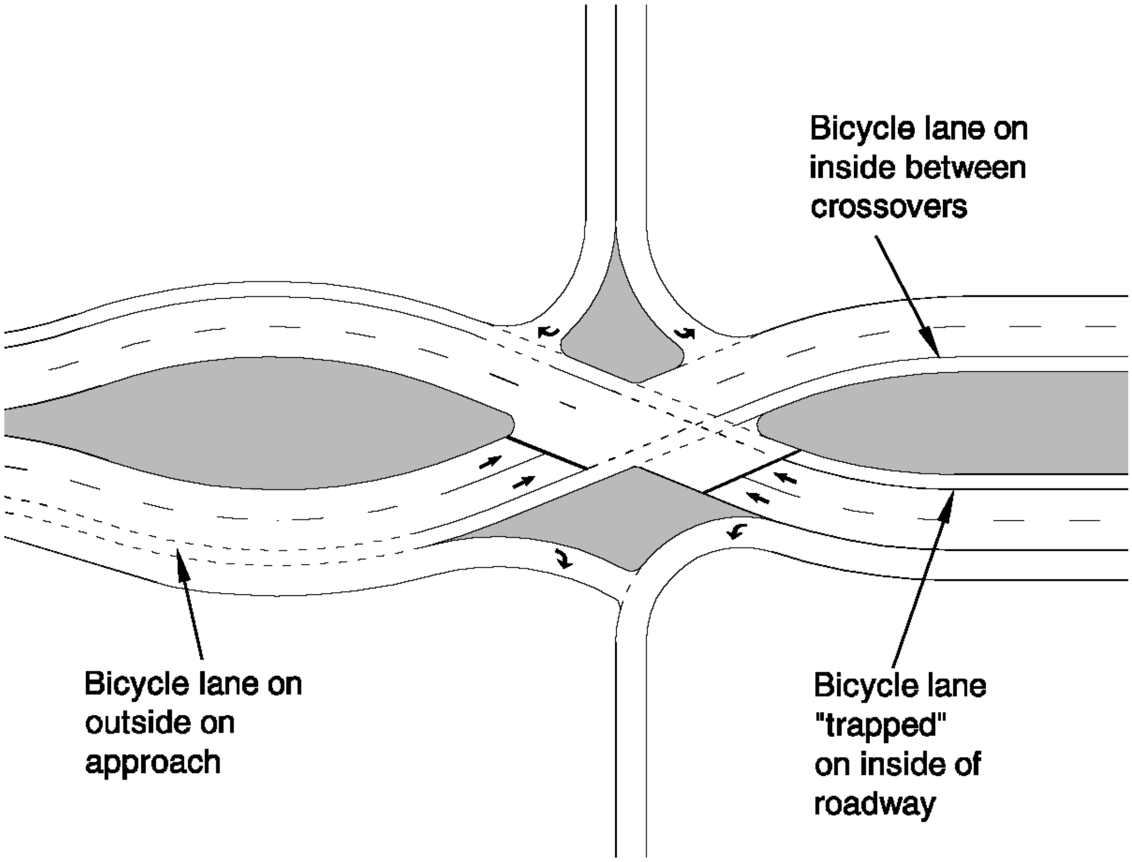


Figure E-26. Schematic for bicycle lane placement on right side of vehicular traffic

Access Management

From an access management standpoint, a DDI intersection provides full access control through the interchange. A traffic simulation should be conducted to determine the impacts and any needed mitigation for adjacent and nearby intersections in the corridor. Some of the possible disadvantages of a DDI include the following:

- Will not allow exit ramp to entrance ramp movements.
- Through movements along the frontage road can only be accommodated via bypass lanes or collector-distributor systems.
- May require modifications to nearby signalized intersections.
- Additional access management control beyond the interchange may be needed to prevent weaving maneuvers.
- May require the relocating or removal of adjacent streets/driveways to accommodate crossover and reverse curves.

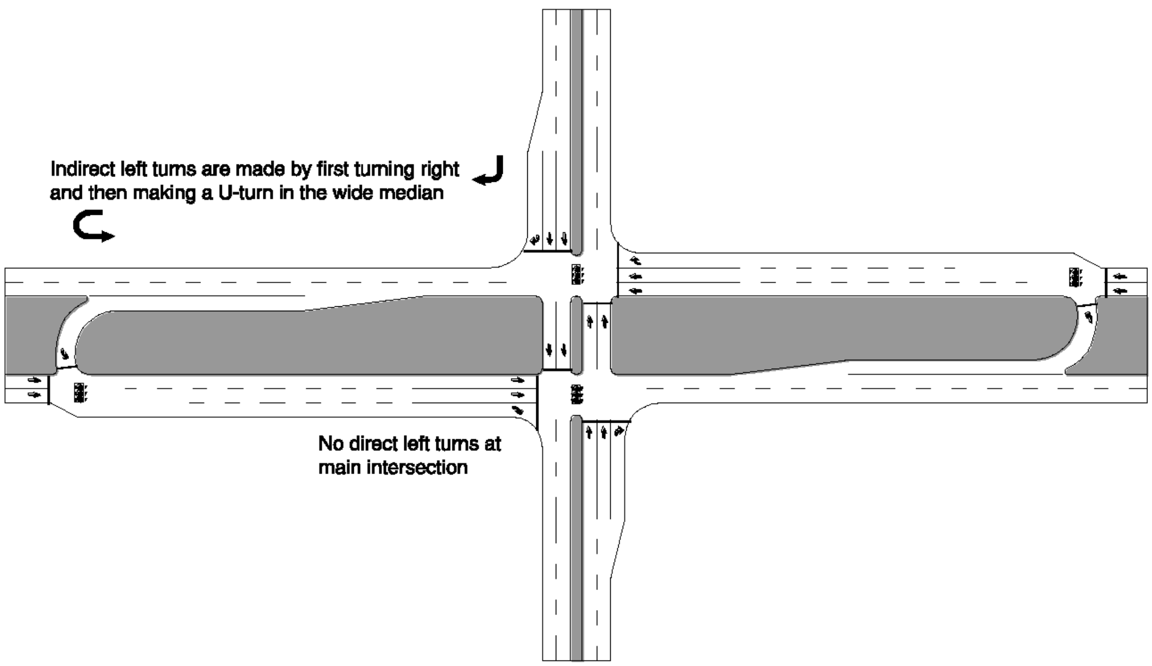
- Removes driveway access for corner development.

Reference the *FHWA Alternative Intersections/Interchanges Informational Report (AIIR)* for additional guidance with respect to access management.

Section 4 — Median U-Turn Intersection (MUT)

Overview

The MUT is an intersection that replaces direct left turns at an intersection with indirect left turns using a U-turn movement in a wide median. The MUT intersection eliminates left turns on both intersecting streets, thus reducing the number of traffic signal phases and conflict points at the main crossing intersection, which results in improved intersection operations and safety. TxDOT has adopted the *FHWA Median U-Turn Informational Guide* as the primary source for MUT Guidelines. The information contained in this Appendix is considered a companion guide and is intended to document TxDOT's suggested approach to MUT design. See Figure E-27 for an example MUT configuration.



Source: FHWA MUT Informational Guide - Exhibit 1-1

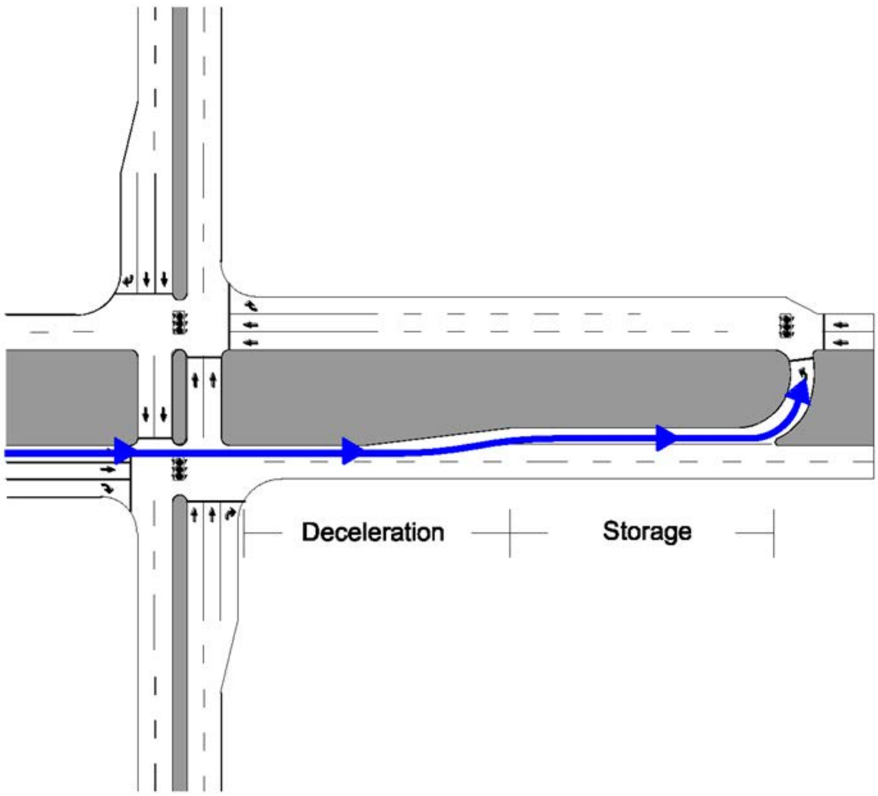
Figure E-27. Example of a MUT Intersection With One Signal in Main Intersection

Some of the documented benefits of a MUT include:

- Removal of left-turn signal phasing which allows the intersection to operate well with a shorter cycle length.
- Reduction of intersection conflict points for both vehicles and pedestrians.
- General increase in safety performance.

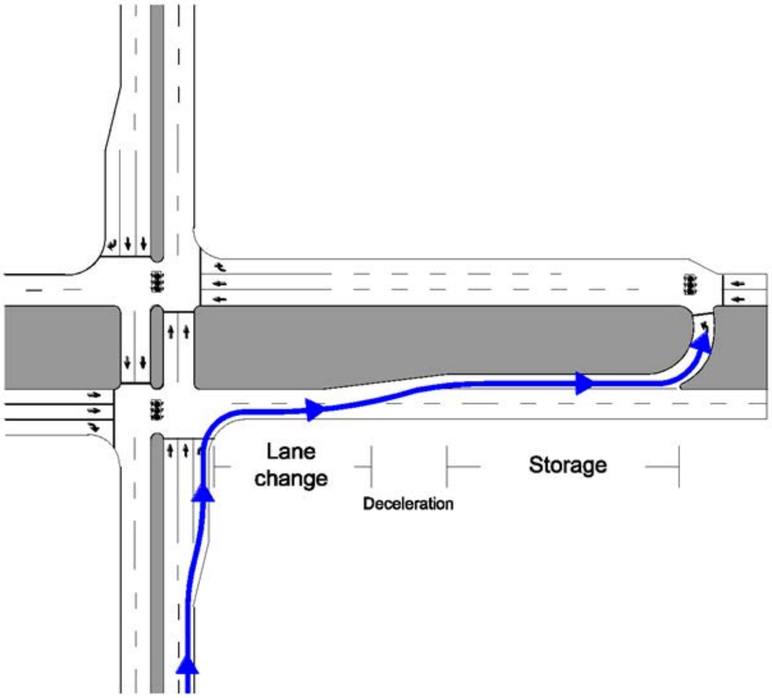
Design Considerations

The distance between the MUT crossovers is central to the proper functioning of the intersection. The AASHTO recommended spacing from the main intersection to the median opening is 400 to 600-ft, although this distance may be modified per guidance in the *FHWA Median U-Turn Informational Guide*. Longer distances are preferred for weaving, but as the distance between the intersection and median opening increases, the delay for the minor road left-turn and through vehicles increases. See Figures E-28 through E-30 for spacing considerations.



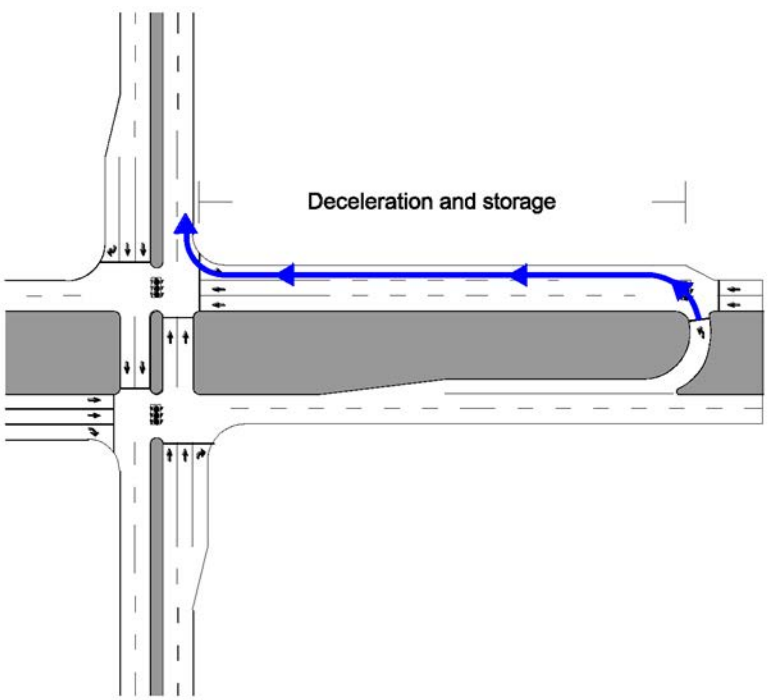
Source: FHWA MUT Informational Guide - Exhibit 7-15

Figure E-28. Spacing Consideration for a Major Street Left Turn Movement



Source: FHWA MUT Informational Guide - Exhibit 7-16

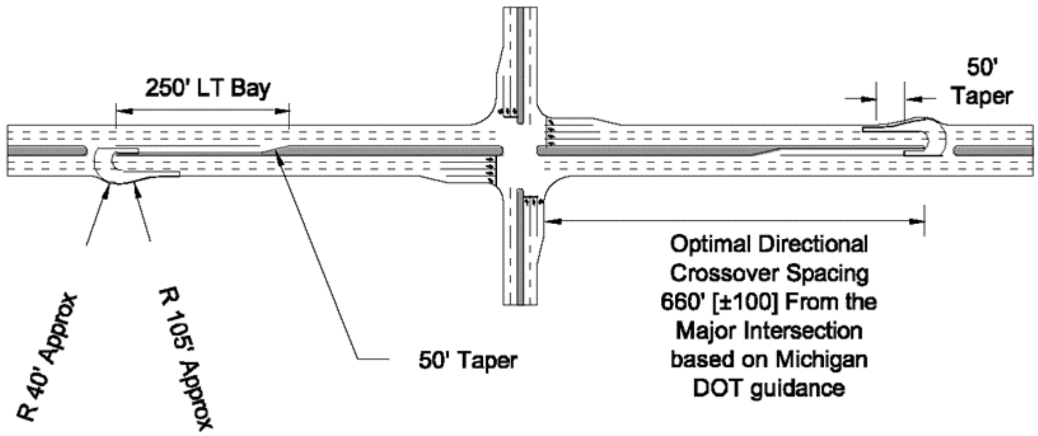
Figure E-29. Spacing Consideration for Minor Street Left Turn Movement



Source: FHWA MUT Informational Guide - Exhibit 7-17

Figure E-30. Spacing consideration for a Right Turn

The MUT median width is a function of the design vehicle (typically a WB-67 in Texas) and the preferred alignment of the design vehicle as it turns into the opposing traffic See Chapter 7 discussion on Median U-Turn Movements and minimum median width needed. For instances where the U-turn can't be completed, a loon may be used to provide additional space (see Figure E-31 as an example application for a longer weaving distance).



Source: FHWA MUT Informational Guide - Exhibit 7-13

Figure E-31. Loon Design Serving a Design Vehicle

Location Considerations

Providing a stop controlled median U-turn location before the main intersection would improve land access between the main intersection and the crossover, route flow, and driver assumptions. However, U-turn movements may conflict with right turn movements. Figure E-32 shows an example of median U-turn locations. Designers must also ensure good intersection sight distances at MUT crossovers by making sure slopes and plantings in the median are cut back beyond the lines of sight.

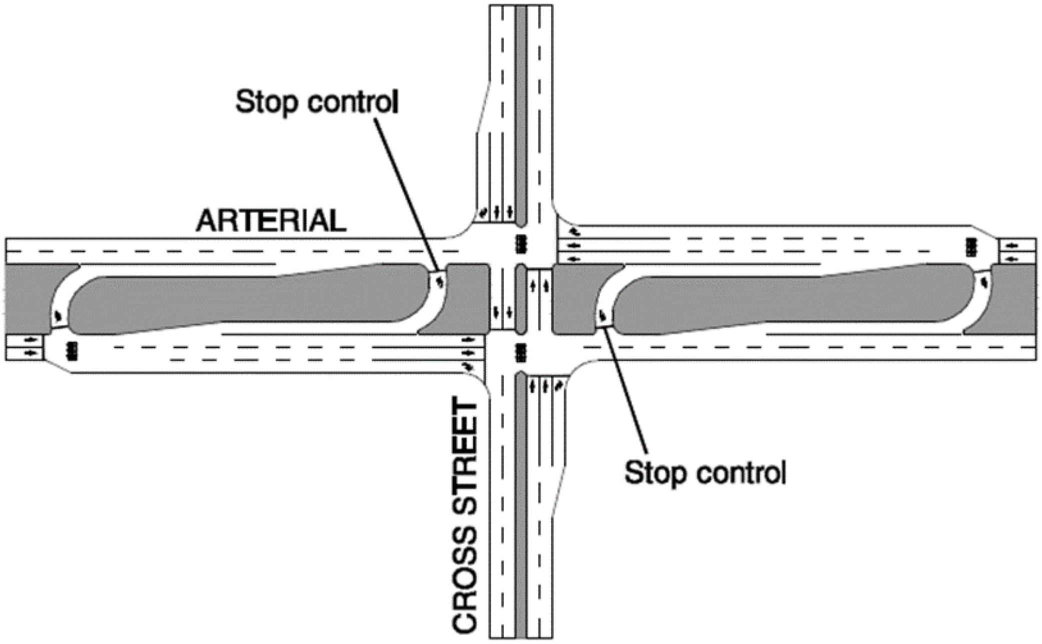
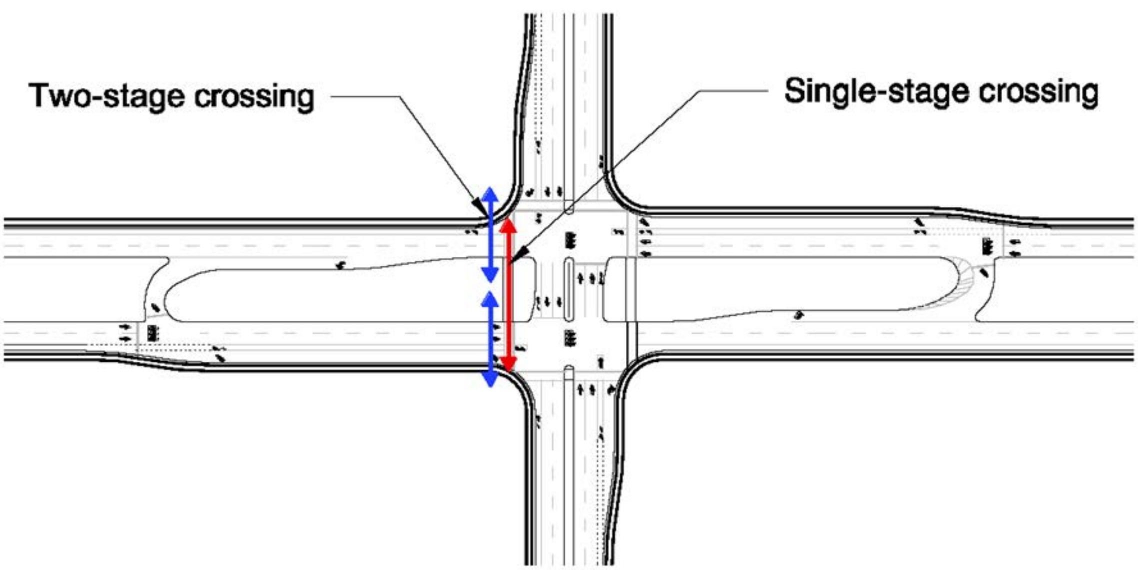


Figure E-32. Stop Controlled Median U-Turn

Pedestrian and Bicyclist Considerations

MUTs have benefits for pedestrians which include a reduction in the number of conflict points and benefits due to two-phase signalization. Single and two-stage pedestrian crossings are shown in Figure E-33.



Source: FHWA MUT Informational Guide - Exhibit 3-4

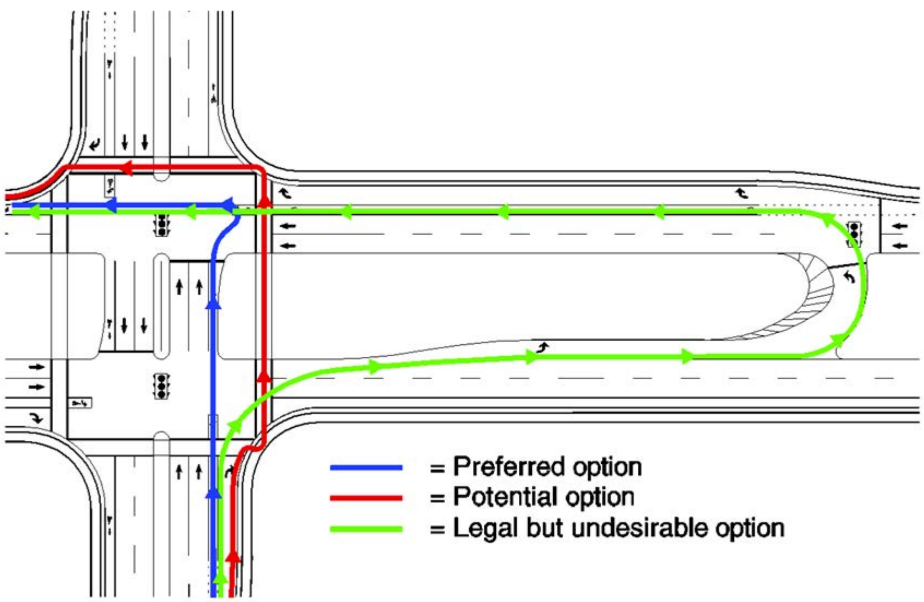
Figure E-33. Single Versus Two-Stage Pedestrian Crossings

Bicyclists have three options for navigating a MUT intersection when making a left turn:

1. Bicyclists making a two-stage left turn (preferred option): Minor street bicyclists approach the intersection on the right and follow the vehicle signal indications. When receiving the green indication, the bicyclists proceed across the intersection and stop in a bicycle turn queue box. When the major street receives a green indication, bicyclists proceed along the major street.
2. Bicyclists follow pedestrian crossing rules: Bicyclists approach the intersection and exit the street to the right and follow the pedestrian (“walk” /” don’t walk”) indications.
3. Bicyclists following vehicle rules: Bicyclists approach the intersection on the right and follow the vehicle signal indications. This option is legally permissible, but undesirable.

Ultimately, a thorough site assessment, an assessment of anticipated bicycle and pedestrian volumes, and an assessment of projected origins and destinations for pedestrians and bicyclists should be conducted to determine the preferred method of movement through the MUT.

Figure E-34 shows the three bicyclist options:



Source: FHWA MUT Informational Guide - Exhibit 3-7

Figure E-34. Left Turn Options for Bicycles

Access Management

From an access management standpoint, possible advantages of a MUT include the elimination of left turns from driveways, and the consolidation of access to U-turn crossover intersections. Possible disadvantages include having to pass twice through intersections, and restricted access between main crossing and U-turn intersections. Reference the *FHWA Alternative Intersections/Interchanges Informational Report (AIIR)* for additional guidance with respect to access management.

Section 5 — Restricted Crossing U-Turn Intersection (RCUT)

Overview

The RCUT, also known as a superstreet intersection, is an at-grade intersection with directional medians such that the minor road traffic must turn right and make a U-turn back to cross or make a left-turn maneuver. The benefits of the RCUT include a significant reduction in intersection conflict points, and a reduction in crash rates and crash severity. TxDOT has adopted the *FHWA Restricted Crossing U-Turn Informational Guide* as the primary source for RCUT Guidelines. The information contained in this Appendix is considered a companion guide and is intended to document TxDOT's suggested approach to RCUT design.

See Figures E-35 through E-37 for examples of RCUTs with signals, without signals, and with merges. For RCUT 3 leg approaches, reference Chapter 7 of the *FHWA Restricted Crossing U-Turn Informational Guide*.

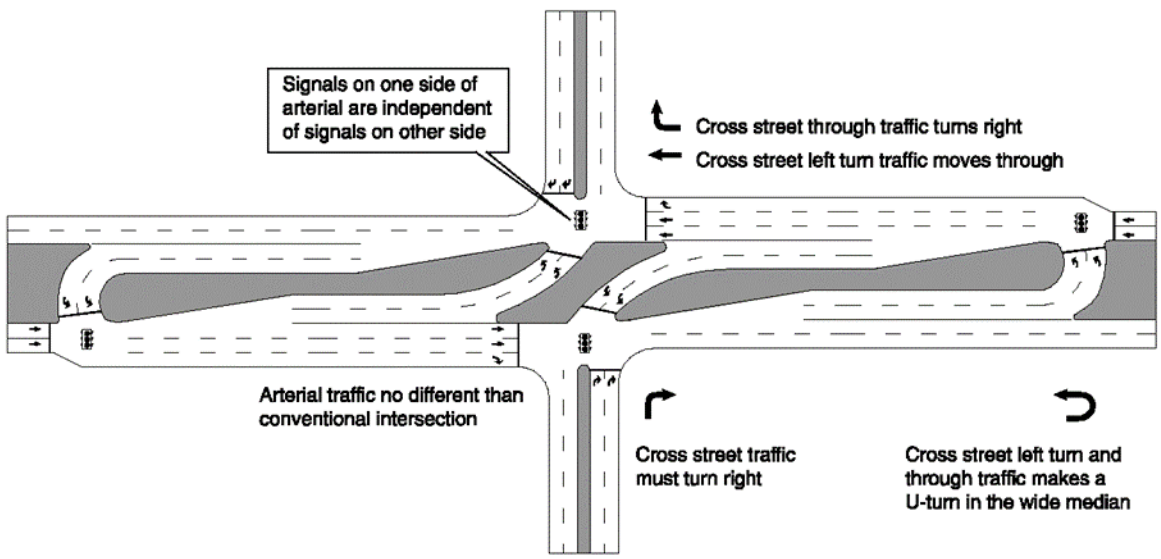
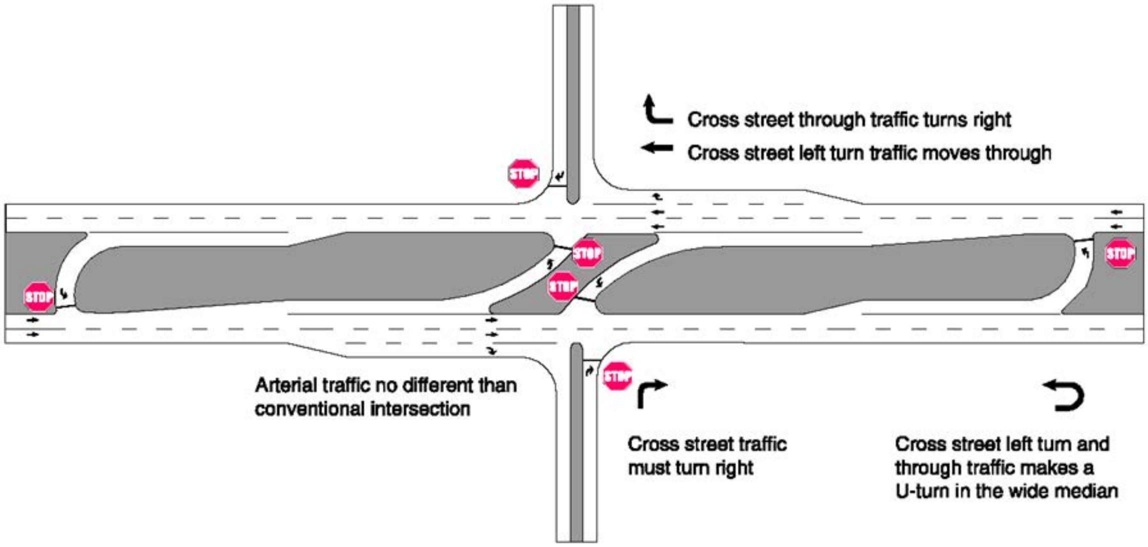
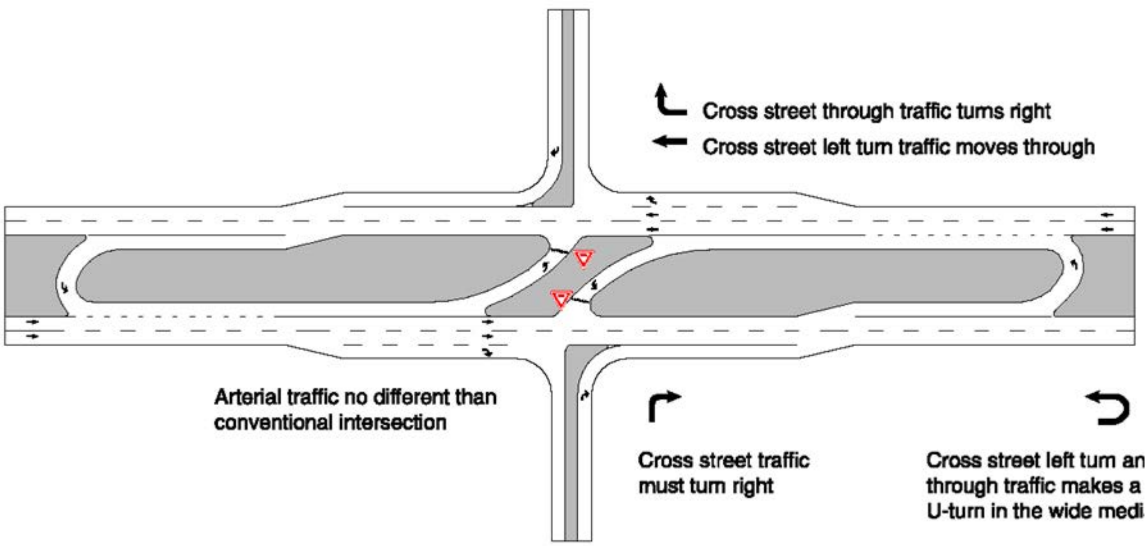


Figure E-35. Example of RCUT Intersection with Signals



Source: FHWA RCUT Informational Guide - Exhibit 1-2

Figure E-36. Example of RCUT Intersection with Stop-Control



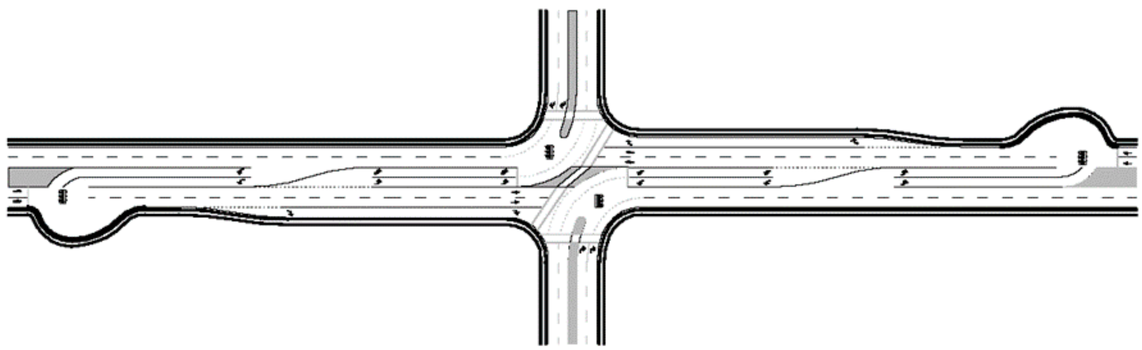
Source: FHWA RCUT Informational Guide - Exhibit 1-3

Figure E-37. Example of RCUT Intersection with Merges

Design Considerations

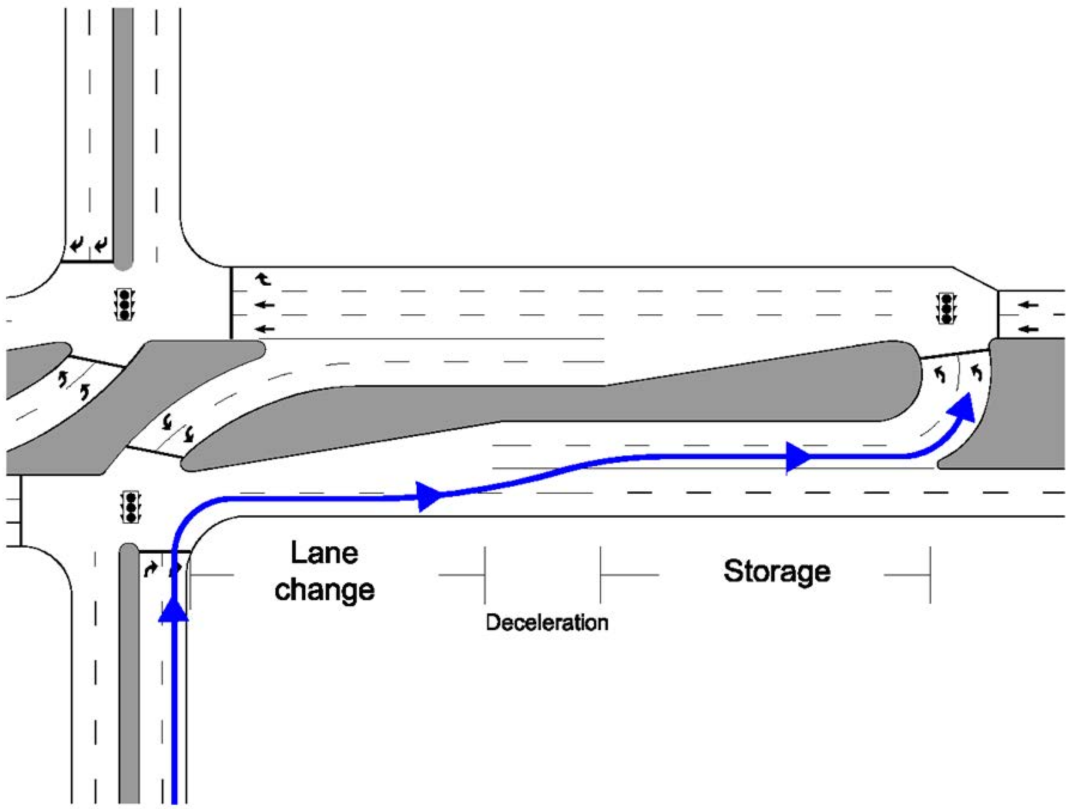
The spacing of the RCUT crossovers from the intersection is central to the proper functioning of the intersection. The typical location for the median opening at an RCUT intersection with stop sign or signal control is 400 to 800-ft. downstream of the minor road.

The median width is a function of the design vehicle (typically a WB-67 in Texas) and the preferred alignment of the design vehicle as it turns into the opposing traffic; see Chapter 7 discussion on Median U-Turn Movements. For instances where the U-turn can't be completed using the existing pavement, a loon type configuration with back-to-back storage bays may be used to provide additional space (see Figure E-38). See Figure E-39 for spacing considerations for a minor street through or left movement. Designers must also ensure good intersection sight distances at RCUT crossovers by making sure slopes and plantings in the median are cut back beyond the lines of sight.



Source: FHWA RCUT Informational Guide - Exhibit 7-19

Figure E-38. RCUT Intersection with Back-to-Back Two-Lane Crossover Storage Bays



Source: FHWA RCUT Informational Guide - Exhibit 7-22

Figure E-39. Spacing Consideration for a Minor Street Through or Left Movement

Intersection Angle

The angle of the side roads to the main lanes of an RCUT intersection can have an impact on traffic operations and the conversion from a conventional intersection to an RCUT intersection. RCUT intersections at an acute angle (less than 90 degrees) generate left turn movements more effectively with the major street as opposed to an obtuse angle (greater than 90 degrees). Figure E-40 demonstrates this concept.

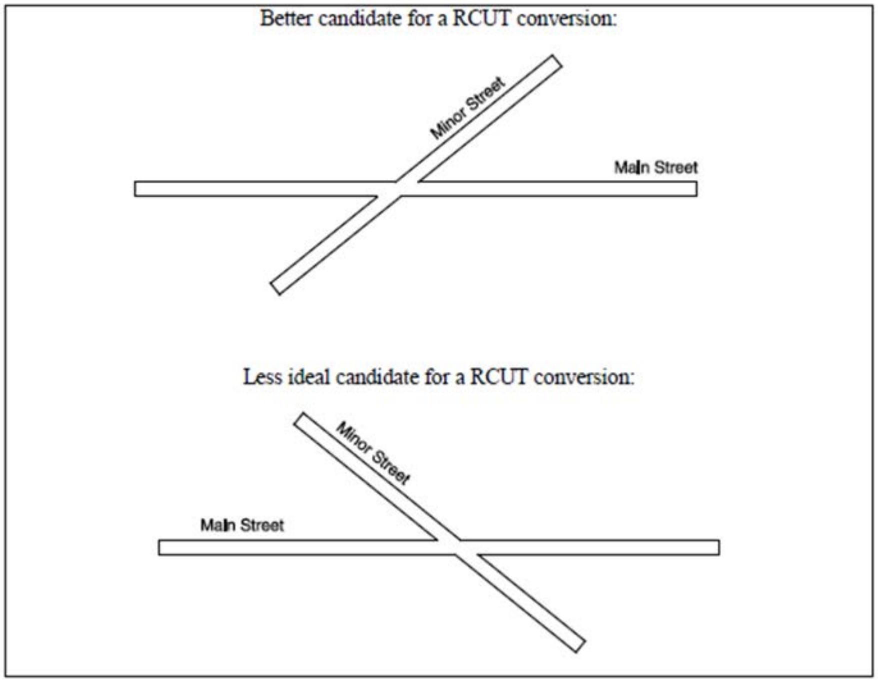
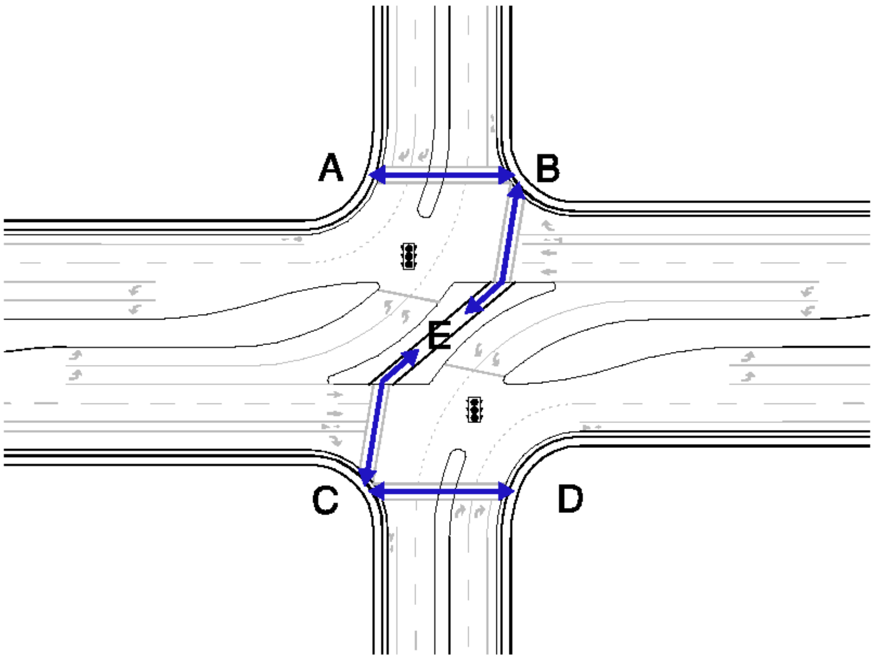


Figure E-40. RCUT Intersection Angle Considerations

Pedestrian and Bicyclist Considerations

RCUTs have benefits for pedestrians, which include a reduction in the number of conflict points and a reduction in cycle lengths (when signalized). In general, an RCUT is better suited for relatively low pedestrian activity or, at least, relatively low volumes of pedestrians crossing the major street. The primary pedestrian crossing pattern is shown in Figure E-41. The *FHWA Restricted Crossing U-Turn Informational Guide* provides guidance for additional pedestrian crossing alternatives.



Source: FHWA RCUT Informational Guide - Exhibit 3-1

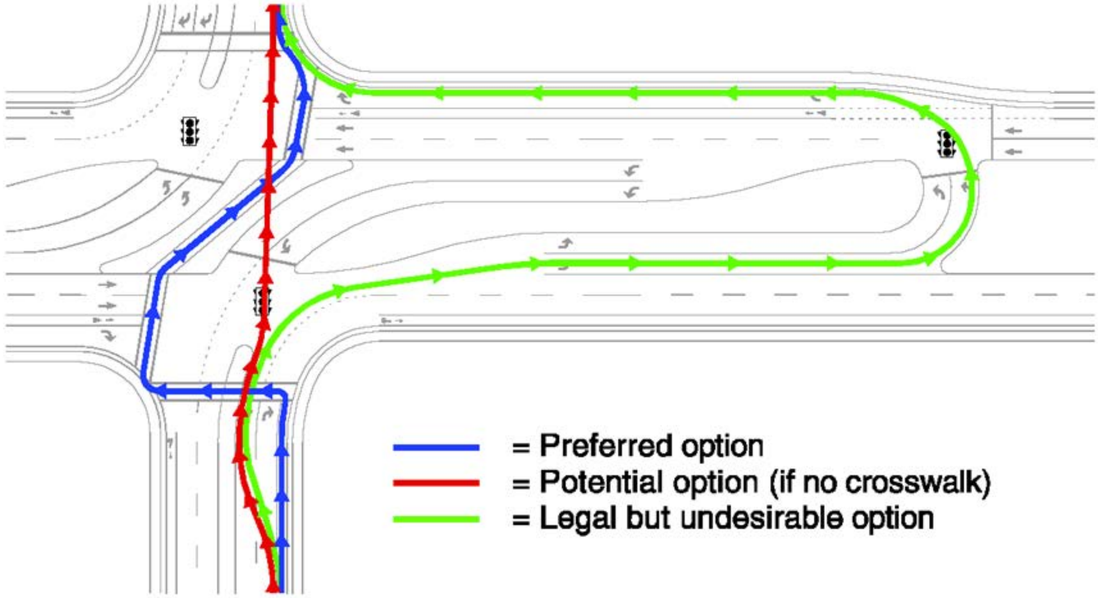
Figure E-41. Pedestrian Movements in a RCUT Intersection

Bicyclists on the major roadway travel through the RCUT the same way they travel through a conventional intersection. The options available for bicyclists approaching on the minor street are:

- Bicyclists follow pedestrian crossing rules (preferred option): Bicyclists approach the intersection and exit the street to the right and follow the pedestrian ("walk" /" don't walk") indications.
- If no crosswalk is available, a potential option is for the bicyclist to pass through/across the channelizing island.
- Bicyclists following vehicle rules: Bicyclists approach the intersection on the right and follow the vehicle signal indications. This option is legally permissible, but undesirable.

Ultimately, a thorough site assessment, an assessment of anticipated bicycle and pedestrian volumes, and an assessment of projected origins and destinations for pedestrians and bicyclists should be conducted to determine the preferred method of movement through the RCUT.

Figure E-42 depicts the three options:



Source: FHWA RCUT Informational Guide - Exhibit 3-12

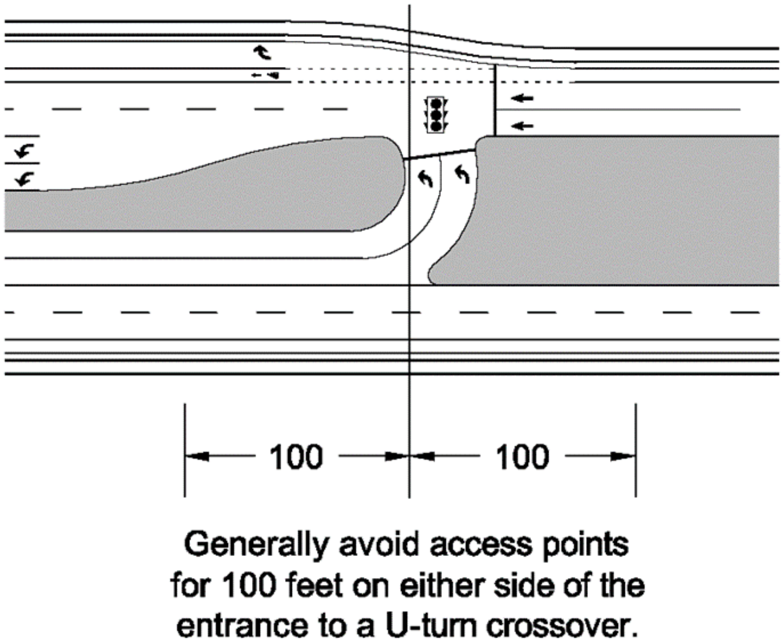
Figure E-42. Minor Street Through Options for Bicycles

Access Management

From an access management standpoint, some of the possible advantages of an RCUT include the following:

- Provides multiple side street locations along the RCUT corridor.
- Allows flexibility for crossover locations to accommodate adjacent driveways and side streets.
- Provides significant progression benefits along the corridor, which can allow for speed control using the signals.

Some of the possible disadvantages include not allowing driveways or side streets near entrances to U-turn crossovers and not having driveways with direct left turns. In general, avoid access points for 100-ft. on either side of the entrance to a U-turn crossover (see Figure E-43). Reference the *FHWA Alternative Intersections/Interchanges Informational Report (AIIR)* for additional guidance with respect to access management.



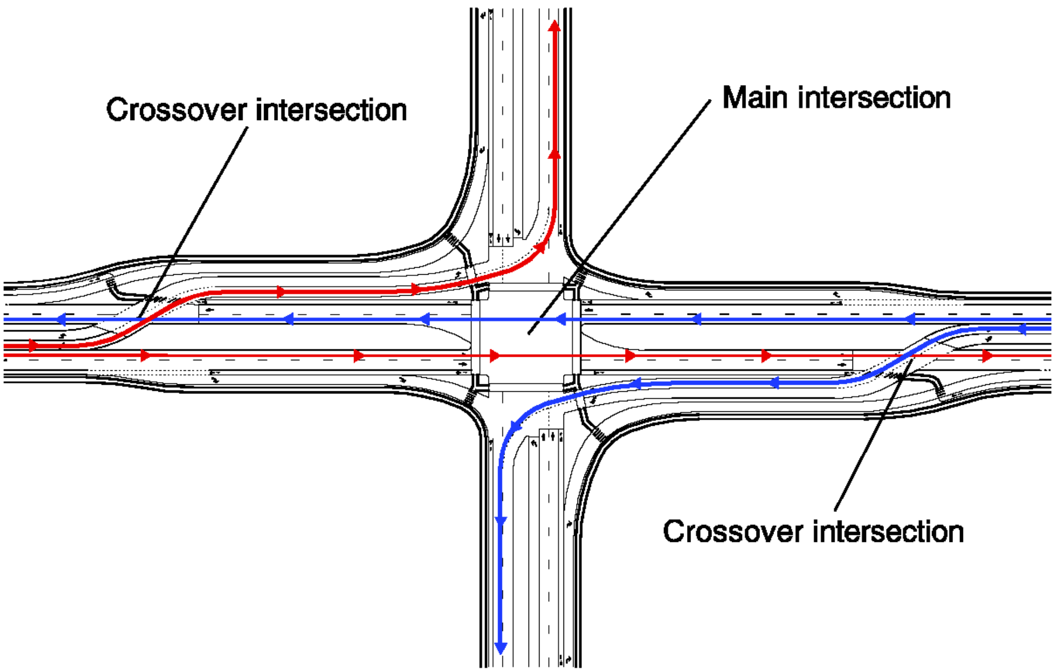
Source: FHWA RCUT Informational Guide - Exhibit 7-14

Figure E-43. Area Near U-turn Crossover Where Access Point Should Be Avoided

Section 6 — Displaced Left Turn Intersection (DLT)

Overview

The DLT, also known as a continuous flow intersection (CFI) is an intersection that relocates one or more left turn movements on an approach to the other side of the opposing traffic flow. This allows left-turn movements to proceed simultaneously with the through movements and eliminates the left-turn phase for this approach. TxDOT has adopted the *FHWA Displaced Left Turn Intersection Informational Guide*[9] as the primary source for DLT Guidelines. The information contained in this Appendix is considered a companion guide and is intended to document TxDOT's suggested approach to DLT design. See Figure E-44 for a typical four-legged DLT with displaced lefts on a major street. (For a DLT 3 leg intersection reference Chapter 7 of the FHWA DLT Informational Guide)



Source: FHWA DLT Informational Guide - Exhibit 1-1

Figure E-44. Four-legged DLT with Displaced Lefts on a Major Street

Benefits of a DLT include a reduction in the number of signal phases with a resultant increase in vehicle throughput. See Figure E-45 for an example of a DLT Intersection with displaced left turns on all approaches.

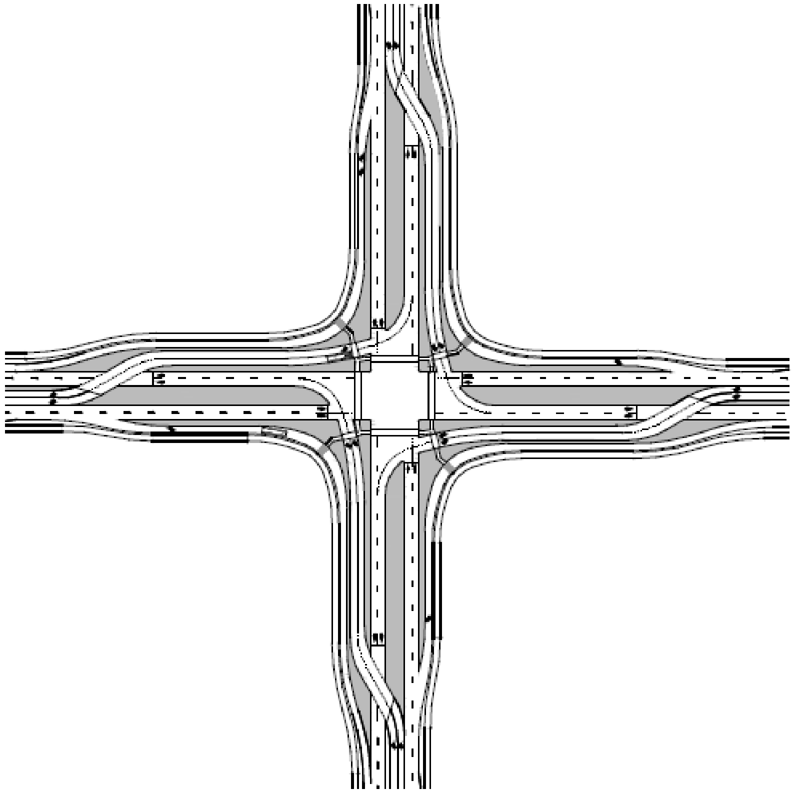
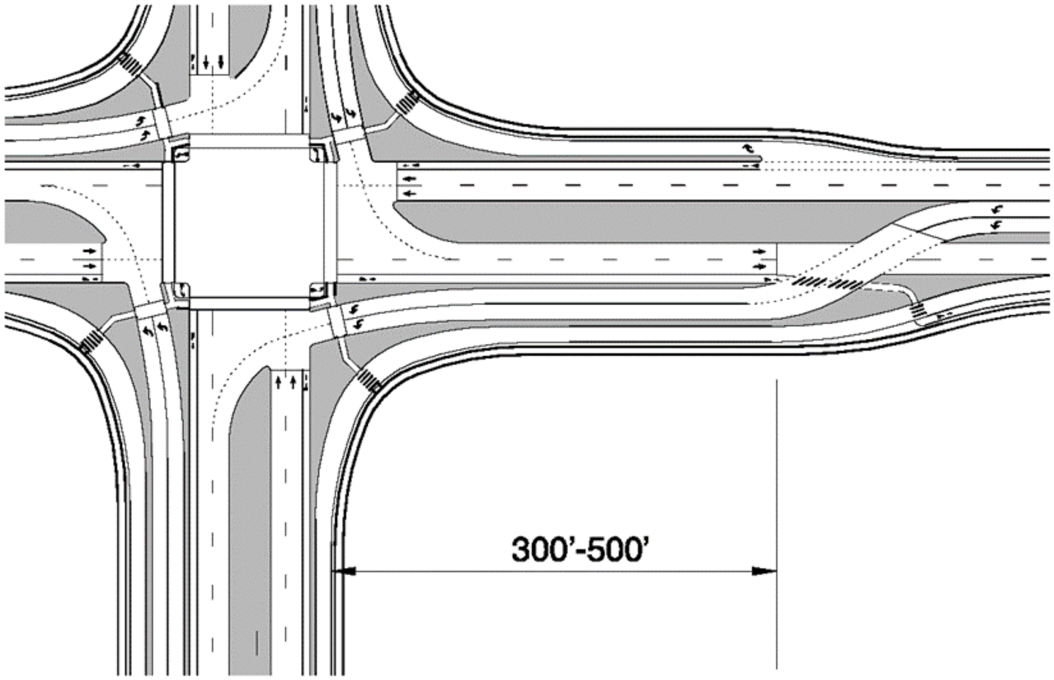


Figure E-45. DLT Intersection with Displaced Left Turns on All Approaches

Design Considerations

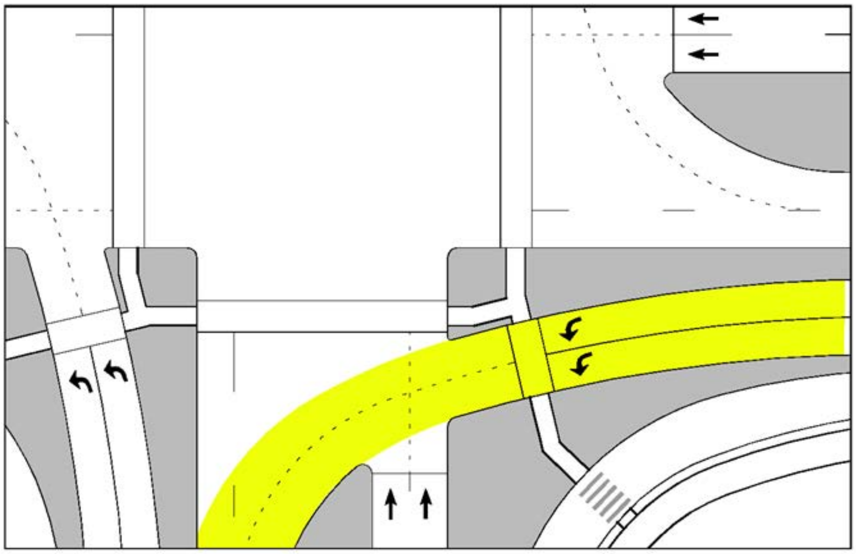
The distance between the main intersection and the crossovers should generally range from 300 to 500-ft. Shorter spacing may result in queue spillback and reduce the ability to clear queues through a single signal cycle phase. Longer spacing may result in greater difficulty in coordinating signal operations. See Figure E-46 for a depiction of typical intersection spacing.



Source: FHWA DLT Informational Guide - Exhibit 7-18

Figure E-46. DLT Typical Intersection Spacing

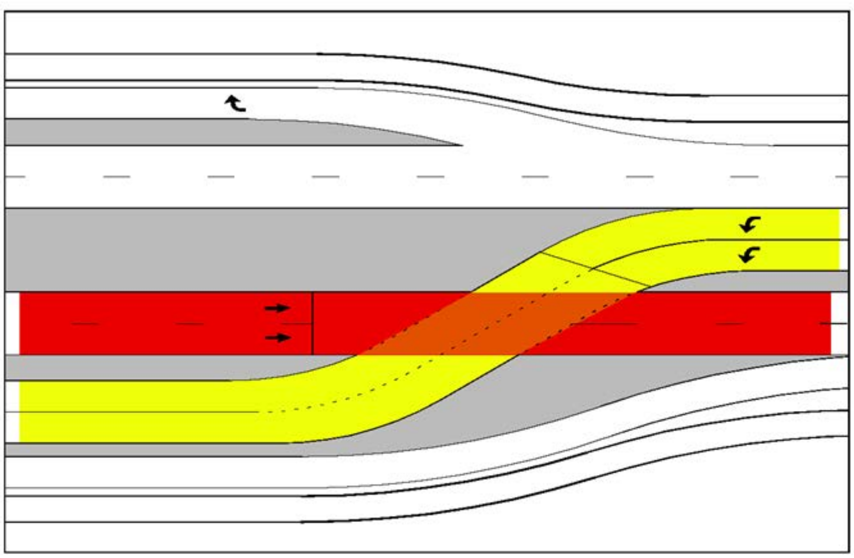
At the main intersection, the primary objective is to provide appropriate turning paths (typically a WB-67 in Texas) for the displaced left turns, considering the interaction with sidewalks. The vehicle paths for the displaced left turns through the main intersection will delineate the curb lines and stop bar locations and determine the width of the overall intersection as shown in Figure E-47.



Source: FHWA DLT Informational Guide - Exhibit 7-2

Figure E-47. DLT Left-Turn Maneuvers

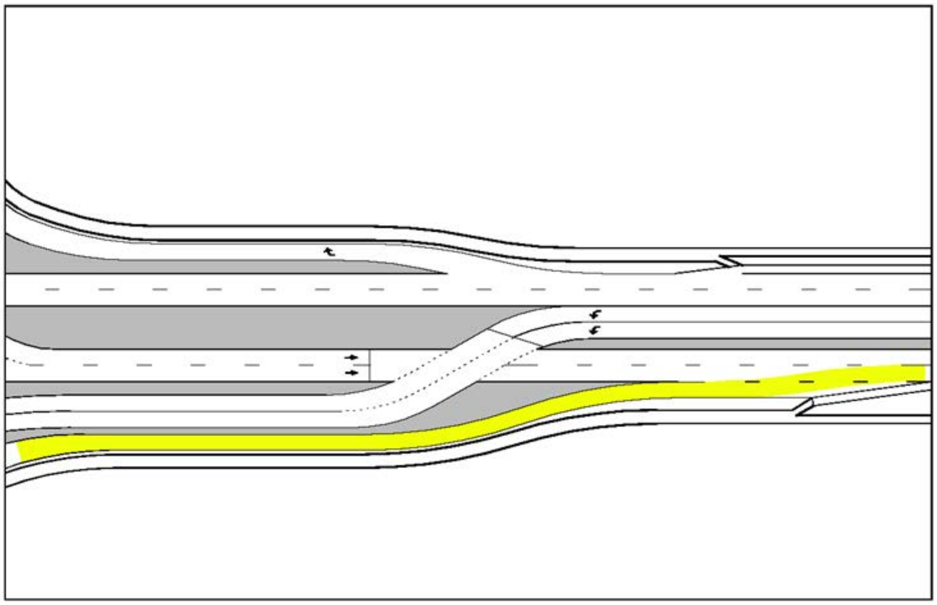
For the crossover intersection, the main objective is to provide a smooth alignment for the traffic and not introduce back-to-back reverse curves along the travel paths. The goal is to align the left turns at the stop bar with the receiving lanes to reflect desirable vehicle path alignment to minimize path overlap as shown in Figure E-48.



Source: FHWA DLT Informational Guide - Exhibit 7-3

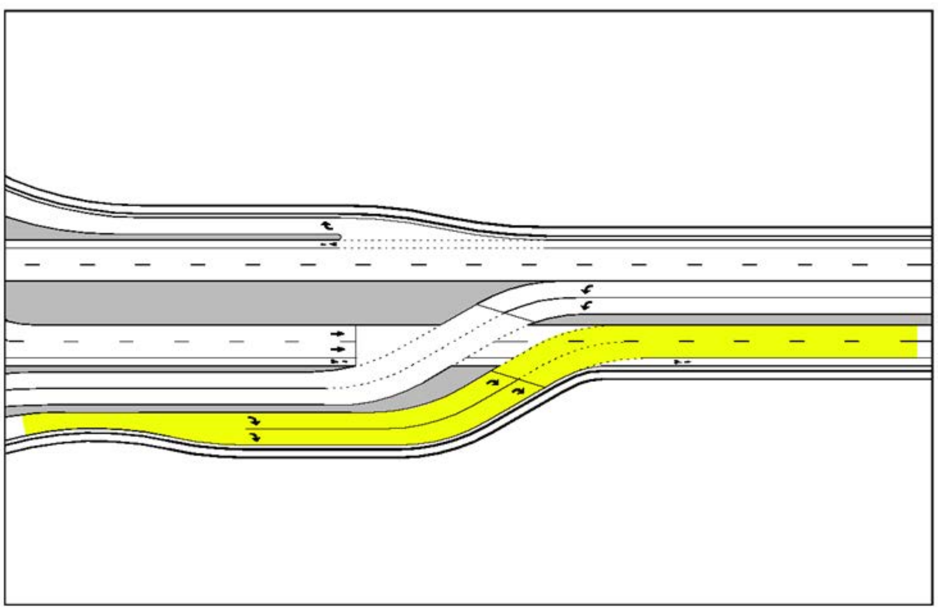
Figure E-48. DLT Crossover Intersection Geometry

Additionally, there are two ways to accommodate the geometry where the right-turn bypass lane joins the cross road through lanes: 1) Provide an additional lane with a downstream merge, as shown in Figure E-49. 2) Signalize the movement and operate it as part of the crossover signal, as shown in Figure E-50.



Source: FHWA DLT Informational Guide - Exhibit 7-4

Figure E-49. Add Lane with a Downstream Lane Merge



Source: FHWA DLT Informational Guide - Exhibit 7-5

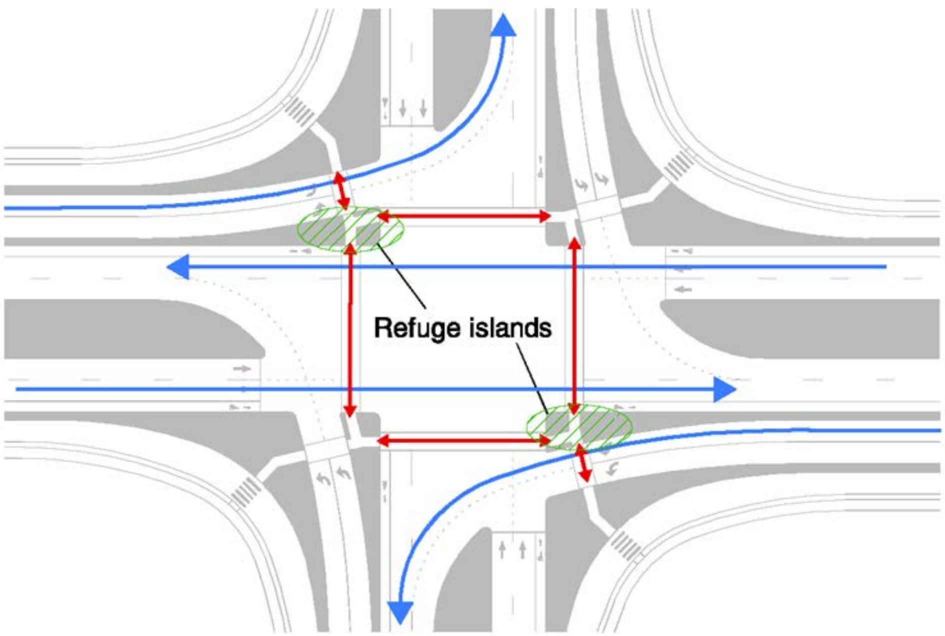
Figure E-50. Signalized Right Turn

Sight Distance

Provide appropriate sight distance and lighting for approaching motorists to see activity at the crosswalk, as well as sight distance for pedestrians to see oncoming traffic. Stopping sight distance for the approaching motorists and sight distances for the pedestrians approaching the potential oncoming automobiles must be clear of obstructions and provide sufficient visibility for various users.

Pedestrian and Bicyclist Considerations

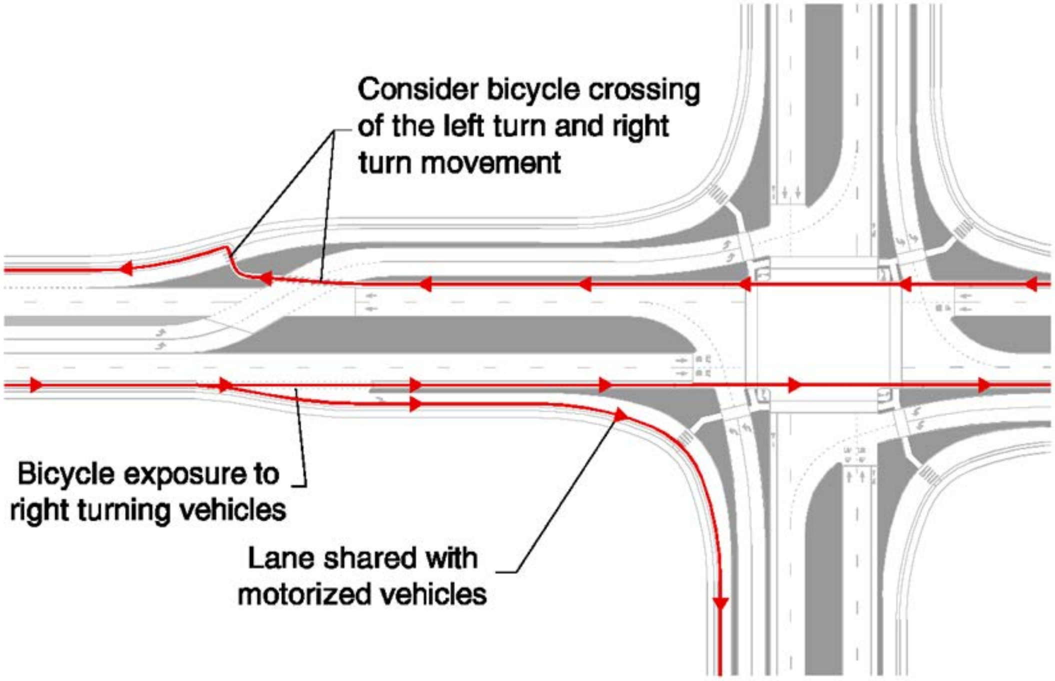
Crosswalks for DLTs are at the same locations that they would be for a conventional intersection. The major street crossing could be made in one or two stages, where the median can be used to provide a refuge for a two-stage crossing. Figure E-51 shows a typical pedestrian crossing with refuge islands.



Source: FHWA DLT Informational Guide - Exhibit 3-4

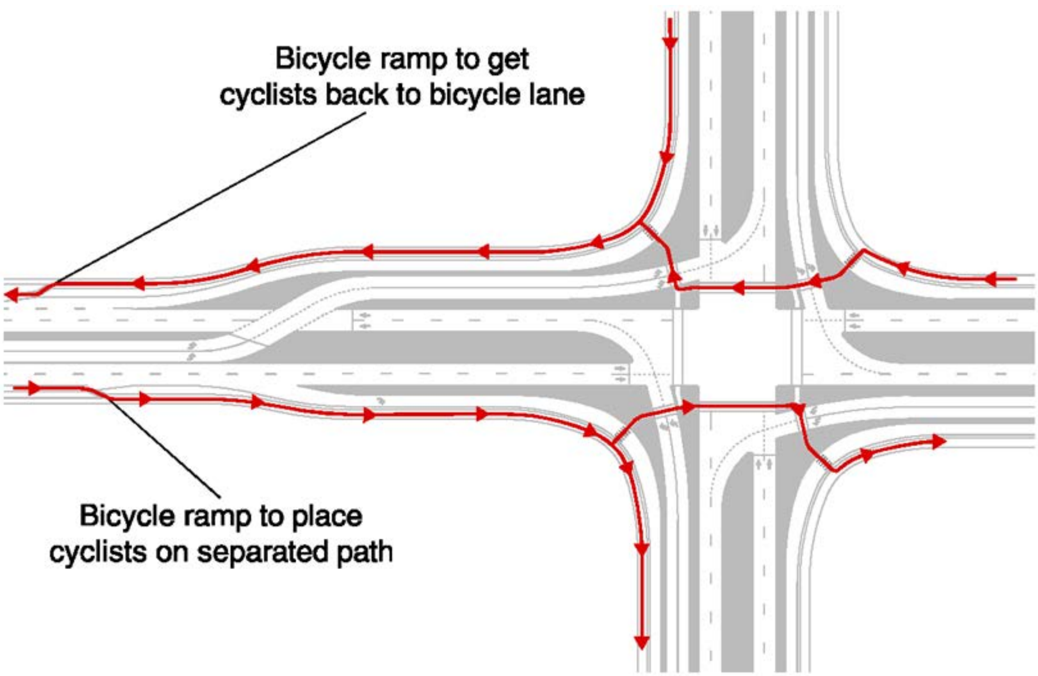
Figure E-51. Refuge Islands Between Left-Turn and Through Lanes

There are various options for bicyclists to use at a DLT; Figures E-52 and E-53 show options for bicycle through movements on and off street, respectively, and Figures E-54 and E-55 show options for bicycle left-turn movements on and off street, respectively. Ultimately, a thorough site assessment, an assessment of anticipated bicycle and pedestrian volumes, and an assessment of projected origins and destinations for pedestrians and bicyclists should be conducted to determine the preferred method of movement through the DLT



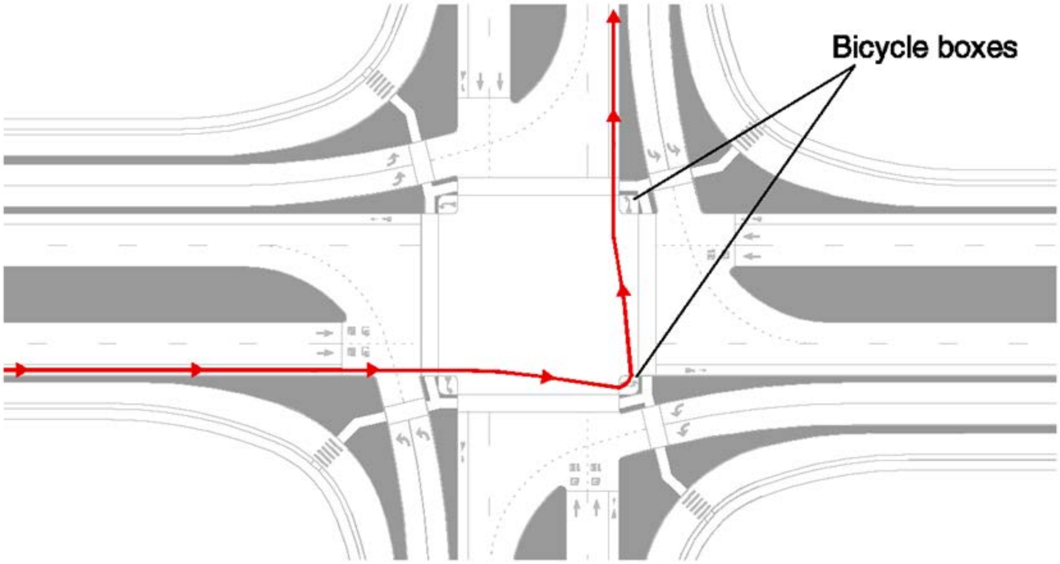
Source: FHWA DLT Informational Guide - Exhibit 3-10

Figure E-52. Accommodating On-Street Bicycles Through a DLT Intersection



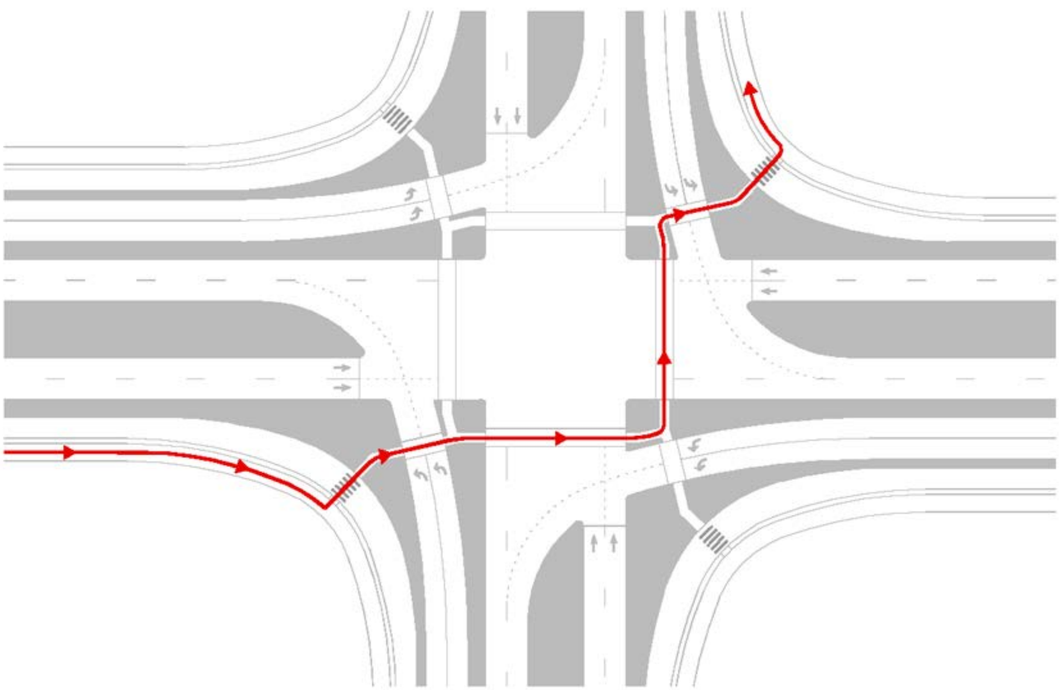
Source: FHWA DLT Informational Guide - Exhibit 3-11

Figure E-53. Accommodating Off-Street Bicycles Through a DLT Intersection



Source: FHWA DLT Informational Guide - Exhibit 3-12

Figure E-54. Accommodating On-Street Left-Turning Bicycles with a Bicycle Box Through a DLT Intersection



Source: FHWA DLT Informational Guide - Exhibit 3-13

Figure E-55. Accommodating Off-Street Left-Turning Bicycles Through a DLT Intersection

Access Management

Maintaining or providing access to homes and businesses near a DLT intersection can be accomplished using frontage roads; however, the following operational impacts may result: Weaving movements in and out of driveways or U-turns (at the main or adjacent intersections) may be in conflict. DLT intersection implementation typically restricts access to parcels situated in the quadrants of the main intersection. Access to these parcels can be accommodated via right-in/right out from the channelized right-turn lanes. Reference the *NCHRP Report 420 "Impacts of Access Management Techniques"* for additional guidance for access management for DLT intersections.

Section 7 — References

1. A Policy on Geometric Design of Highways and Streets, American Association of Highway and Transportation Officials, 2011.
2. ADA Standards for Accessible Design, Americans with Disabilities Act, 2010.
3. Texas Manual on Uniform Traffic Control Devices, Texas Department of Transportation, 2011.
4. NCHRP Report 672: Roundabouts an Informational Guide, 2nd Edition, National Cooperative Highway Research Program, 2010.
5. NCHRP Synthesis 488: Roundabout Practices, National Cooperative Highway Research Program, 2016.
6. FHWA Diverging Diamond Interchange Informational Guide, 2014. (Report No. FHWA-SA-14-067)
7. FHWA Median U-Turn Intersection Informational Guide, 2014. (Report No. FHWA-SA-14-069)
8. FHWA Restricted Crossing U-Turn Intersection Informational Guide, 2014. (Report No. FHWA-SA-14-070)
9. FHWA Displaced Left Turn Intersection Informational Guide, 2014. (Report No. FHWA-SA-14-068)
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