Roadway Design Manual

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Texas Department of Transportation

Manual Notice 2024-1

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Purpose

The *Roadway Design Manual* has been revised, reorganized, and reformatted to update to the current national guidance, national standards, state-of-the-practice, recent transportation research, recent transportation policies, and new topics.

Contents

A correspondence Table reflecting the reorganization of the Manual is below:

RDM (Update) (Chapter)	Current RDM (REV. 2) (Chapter)
Chapter 1 (General Guidance)	Chapter 1 (Design General)
Chapter 2 (Performance Based Practical Design) - New Chapter/Topic	N/A
Chapter 3 (Context and Facility Type Considerations) - New Chapter/Topic	N/A
Chapter 4 (Basic Design Criteria)	Chapter 2 (Basic Design Criteria)
Chapter 5 (Local Roadways 4R)	Chapter 3 (4R Design Criteria)
Chapter 6 (Collectors 4R)	Chapter 3 (4R Design Criteria)
Chapter 7 (Arterials 4R)	Chapter 3 (4R Design Criteria)
Chapter 8 (Freeways 4R)	Chapter 3 (4R Design Criteria)
Chapter 9 (Mobility Corridor Facilities 5R)	Chapter 8 (5R Design Criteria)
Chapter 10 (3R Design Criteria)	Chapter 4 (3R Design Criteria)
Chapter 11 (2R Design Criteria)	Chapter 5 (2R Design Criteria)
Chapter 12 (Special Facilities)	Chapter 6 (Special Facilities)
Chapter 13 (Intersections)	Chapter 2, Section 4 & 6, Chapter 3, & Appendix D (Right Turn Slip Lane Design Guidelines)

Chapter 14 (Alternative Intersection and Interchange Design)	Appendix E (Alternative Intersections and Interchanges)
Chapter 15 (Grade Separations and Interchanges)	Chapter 3 (4R Design Criteria)
Chapter 16 (Driveways)	Appendix C (Driveway Design Guidelines)
Chapter 17 (Roadside Safety Design and Roadside Safety Hardware)	Chapter 7, Section 1, and Appendix A (Longitudinal Barriers)
Chapter 18 (Bicycle Facilities)	Chapter 6, Section 4
Chapter 19 (Pedestrian Facilities)	Chapter 7, Section 3
Chapter 20 (Motorcyclist Design Consideration) - New Chapter/Topic	N/A
Chapter 21 (Texas Highway Freight Network, Texas Trunk System, NHS, and STRAHNET)	Chapter 3 (4R Design Criteria)
Chapter 22 (Transit Facilities) - New Chapter/Topic	N/A
Chapter 23 (Temporary Traffic Control) - New Chapter/Topic	N/A
Chapter 24 (Other Design Elements)	New Material + Chapter 7, Section 2 (Fencing)
Chapter 25 (Index)	N/A
Appendix A (Minimum Values for Design Exceptions)	Respective Minimum criteria values from applicable Chapters
Appendix B (Minimum values for Design Waivers)	Respective Minimum criteria values from applicable Chapters
Appendix C (List of References)	N/A

Roadway Design Manual Format:

Chapter 1 (General Guidance) describes the design elements that require Design Exceptions, Design Waivers, Design Variances, and the Texas Highway Freight Network's Design Deviations. Also includes references for schematic design, access control, preliminary engineering, and maintenance in design.

Chapter 2 (Performance Based Practical Design) discusses the concepts and application of performance-based practical design. Also added guidance from recent research NCHRP 785 (Performance-Based Analysis of Geometric Design of Highways and Streets) and NCHRP 839 (A Performance-Based Highway Geometric Design Process).

Chapter 3 (Context and Facility Type Considerations) describes the five context classifications (urban, suburban, rural, rural town, & urban core) along with considerations for all modes of users and traffic networks. Context approach brought into alignment with AASHTO Greenbook and recent NCHRP research.

Chapter 4 (Basic Design Criteria) presents basic design criteria. Portions of this section will have application to all projects to varying degrees. The chapter discusses speed, terrain, traffic

characteristics, sight distance, horizontal and vertical alignment, combination of horizontal & vertical alignment, cross-sectional elements, drainage facilities, older drivers & pedestrians, and roadway intersecting department projects. The dimensions given in this chapter will be referenced for most of the roadway classifications. Some notable updates include a superelevation table for the proper selection of superelevation methodology as a function of context and/or functional classification. Also, a discussion on the concept of Target Speed.

Chapter 5 (Local Roadways 4R) describes target design criteria (4R) for Local roads and streets (i.e., design speed, lane width, shoulder width, speed change lane width, median width, horizontal curve radius, superelevation rate, etc.) as a function of the Context.

Chapter 6 (Collectors 4R) describes target design criteria (4R) for Collector roads and streets (i.e., design speed, lane width, shoulder width, speed change lane width, median width, horizontal curve radius, superelevation rate, etc.) as a function of the Context.

Chapter 7 (Arterials 4R) describes target design criteria (4R) for Arterial roads and streets (i.e., design speed, lane width, shoulder width, speed change lane width, median width, horizontal curve radius, superelevation rate, etc.) as a function of the Context.

Chapter 8 (Freeways 4R) describes target design criteria (4R) for freeways (i.e., design speed, lane width, shoulder width, speed change lane width, median width, horizontal curve radius, superelevation rate, etc.). The Freeways chapter includes Interstates, Expressways, and Freeways as a function of the Context.

Chapter 9 (Mobility Corridor Facilities 5R) describes and provides design guidance on mobility corridors (5R) with design speeds of 85 mph to 100 mph (i.e., lane width & number, shoulders, cross slope, vertical clearance, stopping sight distance, grades, horizontal alignment, superelevation, vertical curves, roadside design criteria, ramps and direct connectors, etc.).

Chapter 10 (3R Design Criteria) describes 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 (including roadway tradeoff guidance and cross-section optimization, safety enhancements, frontage roads, bridges & bridge-class culverts, and Super 2 design). Also added guidance from recent research TTI 0-6997 (Guidelines for Implementing Super 2 Corridors in Texas).

Chapter 11 (2R Design Criteria) 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 12 (Special Facilities) describes special facility design criteria. Special facilities may include off-system bridge projects, historical roadways or structures, and park roads. 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 13 (Intersections) discusses types of intersections and design criteria and considerations for the design of intersections as well as the design of right turn slip lanes.

Chapter 14 (Alternative Intersection and Interchange Design) 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), Displaced Left Turn Intersections (DLT), and Continuous Green T-Intersection (CGT).

Chapter 15 (Grade Separations and Interchanges) discusses grade separations and interchanges. Describes design criteria for ramps and direct connectors. Added guidance from recent research RTI 0-7047 (Establish Guidelines for Designing Auxiliary Lanes on Frontage Roads). The guidance from RTI 0-7047 provides guidance on merge auxiliary lanes, weaving auxiliary lanes, distance recommendations from an exit ramp to the downstream intersection, left-turn deceleration lanes, right-turn deceleration lanes, and U-turn/turnaround deceleration lanes. Also added guidance from recent research NCHRP 15-56 (Selecting Ramp Design Speeds Vol. 1 & 2).

Chapter 16 (Driveways) discusses design criteria and considerations for the design of driveways.

Chapter 17 (Roadside Safety Design and Roadside Safety Hardware) discusses the design of roadside safety and roadside safety hardware. Topics include general applications of conditions for roadside barriers, types of roadside barriers (Rigid, Semi-rigid, flexible), metal beam guard fence guidance, length of need methodology, Median barrier, crash cushion attenuators, and emergency turnarounds.

Chapter 18 (Bicycle Facilities) describes design criteria and considerations for the design of bicycle facilities.

Chapter 19 (Pedestrian Facilities) describes design criteria and considerations for the design of pedestrian facilities. This includes updates due to the recently adopted PROWAG criteria.

Chapter 20 (Motorcyclist Design Consideration) discusses motorcyclist design considerations such as pavement features, roadside features, highway features, and motorcycle safety during construction.

Chapter 21 (Texas Highway Freight Network, Texas Trunk System, NHS, and STRAHNET) describes the policy for highways on the Texas Highway Freight Network, Texas Trunk System, NHS, and STRAHNET.

Chapter 22 (Transit Facilities) discusses the design of transit facilities for buses and trains as it pertains to vehicular roadways such as Right of Way requirements (i.e., passenger platforms, intersections, transit centers, and dedicated transitways) and Roadway Design (i.e., lane widths, profiles, vertical clearances, acceleration/deceleration, and transit stops).

Chapter 23 (Temporary Traffic Control) discusses design criteria and considerations for the design of temporary traffic control (TTC) (i.e., purpose of TTC design, safety in TTC design, federal regulations, duration, typical sections, shoulder widths, lane widths, bike & ped requirements, property access requirements, temporary geometry, and example problems).

Chapter 24 (Other Design Elements) discusses roadway design considerations for fencing, illumination, signing, pavement marking, intelligent transportation systems (ITS), and wildlife crossings.

Appendix A contains tables of the minimum design values, that if not met, would require a design exception as described in Chapter 1.

Appendix B contains tables of the minimum design values, that if not met, would require a design waiver as described in Chapter 1.

Appendix C contains the source references.

Significant Updates to the RDM:

- Performance Based Practical Design
- Context Classification
- Speed (Design, Operating, Posted, & Target): a newer emphasis on Target Speed that is the operating speed that the designer intends for drivers to use. In mixed-use urban areas, the target speed is the highest speed at which vehicles should operate on a low-speed thoroughfare in a specific context, consistent with the level of multimodal activity generated by adjacent land uses, to provide both mobility for motor vehicles and a desirable environment for pedestrians, bicyclists, and public transit users. In general, the Target speed for rural roadways should be on the higher end of the design speed range.

- Target Design Criteria (4R) Local Roads (Ch. 5), Collector Roads (Ch. 6), Arterials (Ch. 7), and Freeways (Ch. 8): Changing to an emphasis on Target values (with the minimum values provided in the Appendices). The Tables now more align with the AASHTO Greenbook, 4R Chapters are categorized by functional classifications and within the Chapters they are delineated by Context (Rural, Rural Town, Suburban, Urban, & Urban Core) for the respective Design Elements (i.e., design speed, travel lane width, shoulder width, median width, etc.), and typical Sections (e.g., urban core typical section).

- 3R Design Criteria (Including Freeways & Interstates and Roadway Tradeoff Guidance & Cross-Section Optimization):
  In 2022 FHWA issued a final rule which will allow State departments of transportation (State DOT) to adopt procedures or design criteria, as approved by FHWA, that enable

the State to undertake 3R projects on Freeways, including Interstate highways, without utilizing design exceptions as long as the 3R procedures or criteria are met. The practice of roadway design is changing to a more performance-based (PBPD), flexible approach, particularly for 3R projects. Rather than focusing on meeting minimal dimensional criteria, 3R projects can be developed based on project-specific conditions and existing and expected future roadway performance. NCHRP 876 in particular presents a rational approach for estimating the cost-effectiveness of including safety and operational improvements in a 3R project. This PBPD approach provides the basis for determining design improvements that should be incorporated into 3R projects. NCHRP 15-50 (companion to NCHRP 876) reviewed the latest safety literature and developed cost-benefit analysis equations for the most common design improvements applied to 3R projects (including spreadsheet tools Tool 1 and Tool 2).

Roadway Tradeoff Guidance and Cross-Section Optimization: TTI research report 0-7035-R1 studied the safety and operational benefits of converting four-lane undivided highways into other cross section alternatives without changing the overall pavement width, or slightly increasing the overall width dependent on available ROW and clear zone needs. The researchers developed a framework based on safety and operational effects that allows the design engineer to select a cross section depending on traffic volume, heavy vehicle mix, existing cross-sectional width, and access-point density. Also added guidance from follow up recent research TTI 0-7136 (Texas Guidelines for Optimizing Roadway Cross-Section).

- Intersections: includes design considerations (i.e., intersection characteristics, functional area, design objectives, user groups, and capacity), types of intersections (i.e., three-leg and four-leg), alignment and profile, intersection sight distance (i.e., case A, B, C, D, E, F, and G), turning roadways, speed change lanes, median openings, right turn slip lanes, minimum turning radii, and railroad highway grade crossings.
- Alternative Intersection and Interchange Design (additional design guidance). Also added recent research from RTI 0-7036 (Roundabouts/Innovative Intersections), NCHRP Synthesis 550 (Alternative Intersection Design and Selection), and NCHRP 948 (Guide for Ped and Bike Safety at Alternative and Other Intersections and Interchanges), and NCHRP 1043 (Guide for Roundabouts) to RDM chapter 14.
- Grade Separations and Interchanges
- Bicycle Facilities (Micromobility): includes a variety of electrical powered micromobility and power-driven mobility devices (i.e., human or electric powered transportation device, including bicycles, scooters, electric assist bicycles, electric scooters and other small, lightweight, wheeled conveyances) that may be operated on a street, sidewalk, or bikeway unless restricted by a jurisdiction. Micromobility limited to top speeds of 30 mph or less and complements transit by providing first/last mile transportation.
- Pedestrian Facilities (PROWAG): On August 8, 2023, the US Access Board published PROWAG to provide guidance. PROWAG provides 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 Pedestrian Facilities chapter has been updated to the August 8, 2023 edition of PROWAG.

- Motorcyclist Design Considerations
- Temporary Traffic Control
- THFN, Texas Trunk System, NHS, and STRHNET
- Transit Facilities

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 March 2026 Letting, and if final Schematic or 30% plans have not been approved by May 31, 2025. The Districts have the option to use these revisions prior to these dates.

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Archives

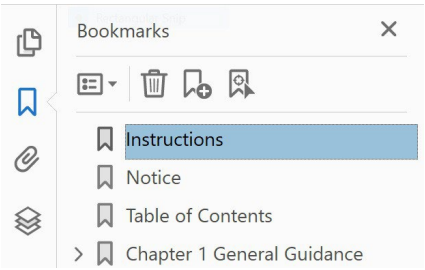
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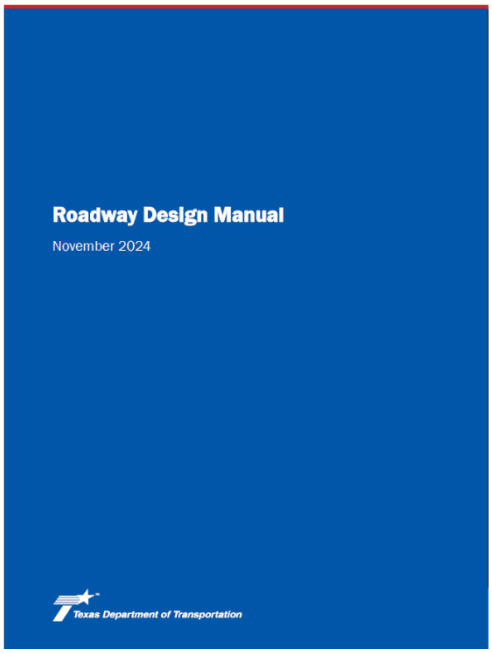
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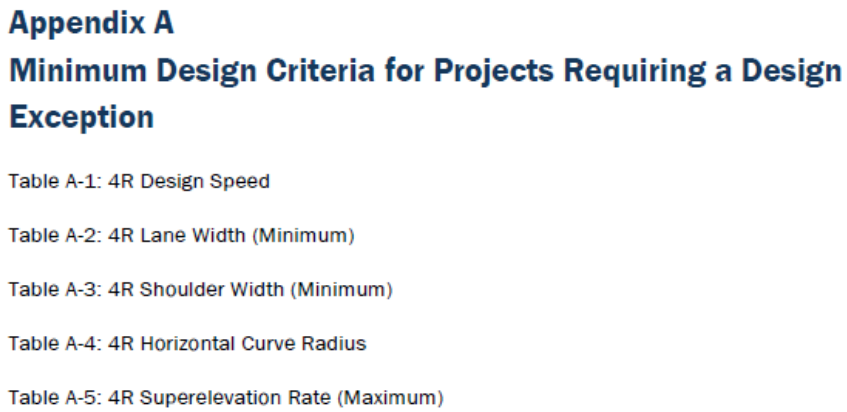
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Reference Links

All external links will link to the Reference Appendix in this eBinder. The Reference Appendix houses all external links in order of appearance. Clicking each link in the Reference Appendix will take you to the external destination.



Contents

Preface xxii

Chapter 1 General Guidance 1-1

1.1 Overview 1-1
1.1.1 Project Objectives and Goals 1-1
1.1.2 Framework for Geometric Design 1-1
1.1.3 Types of Construction 1-2
1.1.4 Roadway Design Manual Format 1-4
1.2 Design Exceptions, Design Waivers, Design Variances, Texas Highway Freeway Network Design Deviations 1-7
1.2.1 Design Exceptions 1-7
1.2.2 Design Waivers 1-13
1.2.3 Design Variances for Pedestrian Facilities 1-18
1.2.4 Texas Highway Freight Network (THFN) Design Deviations 1-18
1.2.5 Approval Process 1-18
1.3 Schematic Development 1-21
1.3.1 Schematic Development Process 1-21
1.3.2 Types of Schematics 1-21
1.3.3 Geometric Schematic Approval Process 1-21
1.4 Access to the Interstate System 1-22
1.5 Preliminary Design 1-25
1.6 Maintenance Considerations in Design 1-26

Chapter 2 Performance-Based Practical Design Concepts 2-1

2.1 Overview 2-1
2.2 Practical Design Process 2-2
2.2.1 What is Practical Design? 2-2
2.2.2 Practical Design Principles 2-2
2.3 Performance-Based Design Process 2-4
2.3.1 What is Performance-Based Design? 2-4
2.3.2 Performance-Based Design Guiding Principles 2-4
2.3.3 Performance-Based Design Process 2-5
2.3.4 Resources for Performance Based Analysis 2-6
2.4 Performance Based Practical Design 2-7
2.4.1 What is Performance Based Practical Design? 2-7
2.4.2 Performance Based Practical Design Application 2-8

2.4.3 What is the Difference between PBPD and Value Engineering? 2-9

Chapter 3 Context and Facility Type Considerations 3-1

3.1 Overview 3-1
3.2 Functional Classification System 3-3
3.3 Context Classification System 3-6
3.3.1 Determining Context: Factors and Process 3-7
3.3.2 Rural Context 3-9
3.3.3 Rural Town Context 3-9
3.3.4 Suburban Context 3-11
3.3.5 Urban Context 3-12
3.3.6 Urban Core Context 3-14
3.4 Roadway Users/Transportation Networks 3-15
3.5 Speed, Mobility and Access Considerations 3-18
3.5.1 Target Speed 3-18
3.5.2 Mobility and Access 3-18
3.6 Implementation of Functional and Context Classification System 3-20

Chapter 4 Basic Design Criteria 4-1

4.1 Overview 4-1
4.2 Speed 4-3
4.2.1 Operating Speed 4-3
4.2.2 Running Speed 4-4
4.2.3 Design Speed 4-4
4.2.4 Target Speed 4-4
4.2.5 Posted Speed 4-5
4.3 Traffic Characteristics 4-6
4.3.1 Traffic Volume 4-6
4.3.2 Design Vehicles 4-8
4.3.3 Control Vehicles 4-9
4.3.4 Highway Capacity 4-11
4.4 Access Management 4-14
4.5 Safety 4-15
4.6 Terrain 4-16
4.6.1 Level Terrain 4-16
4.6.2 Rolling Terrain 4-16
4.6.3 Mountainous Terrain 4-16
4.7 Horizontal Alignment 4-17
4.7.1 General Considerations 4-17

4.7.2 Curve Radius 4-17
4.7.3 Superelevation Rate 4-18
4.7.4 Superelevation Rates on Low-Speed Rural Town, Suburban, Urban and Urban Core Facilities 4-19
4.7.5 Superelevation Rate on Rural and High-Speed Suburban, Urban, and Urban Core Facilities 4-23
4.7.6 Superelevation Transition Length 4-27
4.7.7 Superelevation Transition Placement 4-30
4.7.8 Superelevation Transition Type 4-31
4.7.9 Sight Distance on Horizontal Curves 4-32
4.8 Vertical Alignment 4-34
4.8.1 Grades 4-34
4.8.2 Vertical Curves 4-37
4.8.3 Grade Change without Vertical Curves 4-43
4.8.4 Vertical Alignment at Railroad Crossings 4-44
4.8.5 Vertical Alignment at Intersections 4-44
4.8.6 Vertical Clearance 4-45
4.9 Combination of Horizontal and Vertical Alignment 4-47
4.10 Cross Sectional Elements 4-48
4.10.1 Through Travel Lanes 4-48
4.10.2 Speed Change Lanes 4-48
4.10.3 Shoulders 4-68
4.10.4 Pavement Cross Slope 4-68
4.10.5 Pavement Taper Lengths 4-70
4.10.6 Rumble Strips 4-71
4.10.7 Clear Zone 4-72
4.10.8 Lateral Offset 4-75
4.10.9 Border and Outer Separation 4-76
4.10.10 Right of Way Width 4-79
4.10.11 Roadside Design 4-79
4.10.12 Curbs and Curb/Gutter 4-80
4.10.13 Side Slopes 4-82
4.10.14 Safety Edge 4-84
4.10.15 Medians 4-85
4.10.16 Parking 4-95
4.10.17 Retaining Structures 4-97
4.10.18 Utilities and Easements 4-98

4.11 Sight Distance 4-99
4.11.1 Stopping Sight Distance 4-99
4.11.2 Decision Sight Distance 4-103
4.11.3 Passing Sight Distance 4-104
4.11.4 Intersection Sight Distance 4-107
4.11.5 Sight Distance at Under-Crossings 4-107
4.12 Drainage Facilities 4-109
4.12.1 Design Treatment of Cross Drainage Culvert Ends 4-109
4.12.2 Small Culverts 4-110
4.12.3 Intermediate Culverts 4-111
4.12.4 Large Culverts 4-111
4.12.5 Bridge Class Drainage Culverts 4-112
4.12.6 Parallel Drainage Culverts 4-114
4.12.7 Side Ditches 4-117
4.13 Environment 4-118
4.14 Human Factors 4-119
4.14.1 Older Drivers & Pedestrians 4-119
4.15 Roadway Intersecting Department Projects 4-120
Chapter 5 Local Roads (4R) 5-1
5.1 Design Considerations 5-1
5.1.1 Introduction 5-1
5.1.2 Target Design Values 5-1
5.1.3 Design Speed 5-3
5.1.4 Design Traffic Volumes 5-4
5.1.5 Levels of Service 5-4
5.1.6 Travel Lane Width and Number of Lanes 5-4
5.1.7 Shoulders 5-5
5.1.8 Speed Change Lanes 5-5
5.1.9 Parking Lanes 5-5
5.1.10 Intersections 5-5
5.1.11 Medians 5-5
5.1.12 Curbs 5-5
5.1.13 Cross Slope and Superelevation 5-5
5.1.14 Grades 5-6
5.1.15 Roadside Design 5-6

5.1.16 Structures 5-6
5.1.17 Vertical Clearance 5-6
5.1.18 Bicycle and Pedestrian Facilities 5-6
5.2 Rural Design Element Characteristics 5-7
5.3 Rural Town, Suburban, Urban and Urban Core Design Element Characteristics 5-8
Chapter 6 Collectors (4R) 6-1
6.1 Design Considerations 6-1
6.1.1 Introduction 6-1
6.1.2 Target Design Values 6-1
6.1.3 Design Speed 6-4
6.1.4 Design Traffic Volumes 6-4
6.1.5 Levels of Service 6-4
6.1.6 Travel Lane Width and Number of Lanes 6-5
6.1.7 Shoulders 6-6
6.1.8 Speed Change Lanes 6-7
6.1.9 Parking Lanes 6-7
6.1.10 Intersections 6-7
6.1.11 Medians 6-7
6.1.12 Curbs 6-7
6.1.13 Cross Slope and Superelevation 6-7
6.1.14 Grades 6-7
6.1.15 Roadside Design 6-7
6.1.16 Structures 6-7
6.1.17 Vertical Clearance 6-8
6.1.18 Grade Separations, Interchanges, and Ramps 6-8
6.1.19 Bicycle and Pedestrian Facilities 6-8
6.1.20 Access Control 6-8
6.2 Rural Design Elements 6-9
6.2.1 General Characteristics 6-9
6.2.2 Typical Sections 6-9
6.3 Rural Town Design Elements 6-12
6.3.1 General Characteristics 6-12
6.3.2 Typical Sections 6-12
6.4 Suburban, Urban, and Urban Core Design Elements 6-13
6.4.1 General Characteristics 6-13

6.4.2 Typical Sections 6-14
Chapter 7 Arterials (4R) 7-1
7.1 Design Considerations 7-1
7.1.1 Introduction 7-1
7.1.2 Target Design Values 7-2
7.1.3 Design Speed 7-4
7.1.4 Design Traffic Volumes 7-4
7.1.5 Levels of Service 7-4
7.1.6 Travel Lane Width and Number of Lanes 7-5
7.1.7 Shoulders 7-6
7.1.8 Speed Change Lanes 7-7
7.1.9 Parking Lanes 7-7
7.1.10 Intersections 7-7
7.1.11 Medians 7-7
7.1.12 Curbs 7-7
7.1.13 Cross Slope and Superelevation 7-7
7.1.14 Grades 7-7
7.1.15 Roadside Design 7-7
7.1.16 Structures 7-8
7.1.17 Vertical Clearance 7-8
7.1.18 Grade Separations, Interchanges, and Ramps 7-8
7.1.19 Bicycle and Pedestrian Facilities 7-8
7.1.20 Access Control 7-8
7.2 Rural Design Elements 7-
7.2.1 General Characteristics 7-9
7.2.2 Typical Section 7-9
7.2.3 Transitions to Four-Lane Divided Highways 7-12
7.2.4 Converting Two-Lane Roadways to Four-Lane Divided Facilities 7-13
7.2.5 Converting Two-Lane Roadways to Optimized Roadway Cross-sections 7-14
7.3 Rural Town Design Elements 7-15
7.3.1 General Characteristics 7-15
7.3.2 Typical Section 7-15
7.4 Suburban Design Elements 7-17
7.4.1 General Characteristics 7-17
7.4.2 Typical Section 7-17

7.5 Urban and Urban Core Design Elements 7-20
7.5.1 General Characteristics 7-20
7.5.2 Typical Section 7-20
Chapter 8 Freeways (4R) 8-1
8.1 Design Considerations 8-1
8.1.1 Introduction 8-1
8.1.2 Target Design Values 8-2
8.1.3 Design Speed 8-3
8.1.4 Design Traffic Volumes 8-3
8.1.5 Level of Service 8-4
8.1.6 Travel Lane Width and Number of Lanes 8-4
8.1.7 Shoulders 8-10
8.1.8 Speed Change Lanes 8-10
8.1.9 Medians 8-11
8.1.10 Curbs 8-11
8.1.11 Cross Slope and Superelevation 8-12
8.1.12 Grades 8-12
8.1.13 Roadside Design 8-12
8.1.14 Structures 8-12
8.1.15 Vertical Clearance 8-12
8.1.16 Ramps and Terminals 8-12
8.1.17 Outer Separations and Borders 8-12
8.1.18 Frontage Roads 8-13
8.1.19 Collector-Distributor Roads 8-15
8.1.20 Crossing Facilities 8-18
8.1.21 Access Control 8-18
8.1.22 Managed Lanes and Toll Facilities 8-25
8.1.23 Tunnels 8-29
8.2 Rural Design Elements 8-30
8.2.1 General Design Considerations 8-30
8.3 Suburban Design Elements 8-31
8.3.1 General Design Considerations 8-31
8.4 Urban Design Elements 8-32
8.4.1 General Design Considerations 8-32
8.4.2 Depressed Freeways 8-32

8.4.3 Elevated Freeways 8-32
8.4.4 Ground Level Freeways 8-33
8.4.5 Combination-Type Freeways 8-33
8.4.6 Special Freeway Designs 8-33
8.5 Urban Core Design Elements 8-35
8.5.1 General Design Considerations 8-35
Chapter 9 Mobility Corridor Facilities (5R) 9-1
9.1 Design Considerations 9-1
9.1.1 Introduction 9-1
9.2 Cross Sectional Elements 9-3
9.2.1 Overview 9-3
9.2.2 Lane Width and Number of Lanes 9-3
9.2.3 Shoulders 9-3
9.2.4 Pavement and Cross Slope 9-3
9.2.5 Vertical Clearance 9-3
9.2.6 Stopping Sight Distance 9-3
9.2.7 Grades 9-4
9.2.8 Horizontal Alignment 9-5
9.2.9 Superelevation 9-5
9.2.10 Superelevation Transition 9-8
9.2.11 Vertical Curves 9-9
9.3 Roadside Design Criteria 9-10
9.3.1 Clear Zone 9-10
9.3.2 Side Slopes 9-10
9.3.3 Medians 9-10
9.4 Ramps and Direct Connectors 9-11
9.4.1 Overview 9-11
9.4.2 Design Speed 9-11
9.4.3 Lane and Shoulder Widths 9-12
9.4.4 Acceleration and Deceleration Lengths 9-12
9.4.5 Distance Between Successive Ramps 9-14
9.4.6 Grades 9-14
9.4.7 Cross Section and Cross Slope 9-15
Chapter 10 3R Design Criteria 10-1
10.1 Design Considerations 10-1

10.1.1 Introduction 10-1
10.1.2 Freeways and Interstates 10-2
10.1.3 Pavement Design 10-3
10.1.4 Minimum Geometric Design Values 10-3
10.1.5 Design Values 10-3
10.2 Roadway Tradeoff Guidance and Cross-section Optimization 10-6
10.2.1 Overview 10-6
10.3 Safety Enhancements 10-8
10.3.1 Overview 10-8
10.3.2 Safety Design Evaluation 10-8
10.3.3 Basic Safety Improvements 10-10
10.4 Frontage Roads 10-13
10.4.1 Overview 10-13
10.5 Bridges & Bridge-Class Culverts (BCCs) 10-14
10.5.1 Overview 10-14
10.6 Super 2 Design 10-15
10.6.1 Overview 10-15
10.6.2 Basic Design Criteria 10-18
10.7 3R Project Documentation 10-21
10.7.1 Project-Specific Design Information 10-21
10.7.2 Plan Set Requirements 10-22
Chapter 11 2R Design Criteria 11-1
11.1 Introduction 11-1
11.1.1 Definition 11-1
11.1.2 Upgrading 11-1
11.2 Crash Analysis 11-2
Chapter 12 Special Facilities 12-1
12.1 Introduction 12-1
12.2 Off-System Bridge Replacement and Rehabilitation Projects (Low-Volume) 12-2
12.2.1 Overview 12-2
12.2.2 Design Values 12-2
12.3 Historically Significant Bridges 12-4
12.3.1 Reference for Procedures 12-4
12.4 Texas Parks and Wildlife Department (Park Road (PR) and Park and Wildlife Road (PW)) Projects 12-5
12.4.1 Working Agreements 12-5

Chapter 13 Intersections 13-1
13.1 Overview 13-1
13.2 Design Considerations 13-2
13.2.1 Intersection Characteristics 13-2
13.2.2 Intersection Functional Area 13-4
13.2.3 Design Objectives 13-6
13.2.4 Design Considerations for Intersection User Groups 13-7
13.2.5 Intersection Capacity 13-9
13.3 Types of Intersections 13-10
13.3.1 Three-Leg Intersection 13-12
13.3.2 Four-Leg Intersection 13-15
13.4 Alignment and Profile 13-19
13.4.1 Horizontal Alignment 13-19
13.4.2 Profile 13-19
13.5 Intersection Sight Distance 13-20
13.5.1 Case A-Intersections with no control 13-23
13.5.2 Case B-Intersections with stop control on the minor road 13-23
13.5.3 Case C-Intersections with yield control on the minor road 13-24
13.5.4 Case D-Intersections with traffic signal control 13-26
13.5.5 Case E-Intersections with all-way stop control 13-26
13.5.6 Case F-Left turns from the major road 13-26
13.5.7 Case G-Roundabouts 13-27
13.6 Turning Roadways 13-28
13.7 Speed Change Lanes 13-29
13.8 Median Openings 13-30
13.9 Right Turn Slip Lanes 13-31
13.9.1 Urban Design 13-31
13.9.2 Suburban Design 13-39
13.9.3 Rural Design 13-40
13.10 Additional Intersection Design Considerations 13-41
13.10.1 Minimum Turning Radii 13-41
13.11 Railroad-Highway Grade Crossings 13-47
13.11.1 Horizontal Alignment 13-47
13.11.2 Vertical Alignment 13-47
13.11.3 Crossing Design 13-48
13.11.4 Sight Distance 13-49

Chapter 14 Alternative Intersections and Interchanges 14-1
14.1 Introduction 14-1
14.2 Pedestrian Considerations for Alternative Intersections 14-4
14.3 Intersection Analysis/Intersection Control Evaluation (ICE) 14-5
14.4 Roundabouts 14-6
14.4.1 Overview 14-6
14.4.2 Goals of a Roundabout Design 14-7
14.4.3 Design Process and Design Context 14-8
14.4.4 Design Parameters 14-11
14.4.5 Performance Checks 14-19
14.4.6 Special Considerations 14-28
14.4.7 Preliminary and Final Design 14-34
14.5 Continuous Green T-Intersection (CGT) 14-41
14.5.1 Overview 14-41
14.5.2 Design Considerations 14-42
14.5.3 Sight Distance 14-42
14.5.4 Access Management 14-42
14.5.5 Pedestrian and Bicyclist Considerations 14-42
14.6 Restricted Crossing U-Turn Intersection (RCUT) 14-44
14.6.1 Overview 14-44
14.6.2 Design Considerations 14-46
14.6.3 Intersection Angle 14-48
14.6.4 Pedestrian and Bicyclist Considerations 14-48
14.6.5 Access Management 14-50
14.6.6 Safety 14-51
14.6.7 Operations 14-51
14.6.8 Traffic Control Devices 14-51
14.7 Displaced Left-Turn Intersection (DLT) 14-52
14.7.1 Overview 14-52
14.7.2 Design Considerations 14-54
14.7.3 Pedestrian and Bicyclist Considerations 14-56
14.7.4 Access Management 14-57
14.7.5 Safety 14-58
14.7.6 Operations 14-58
14.7.7 Traffic Control Devices 14-58
14.8 Median U-Turn Intersection (MUT) 14-60

14.8.1 Overview 14-60
14.8.2 Design Considerations 14-61
14.8.3 Pedestrian and Bicyclist Considerations 14-63
14.8.4 Access Management 14-64
14.8.5 Safety 14-65
14.8.6 Operations 14-65
14.8.7 Traffic Control Devices 14-65
14.9 Quadrant Roadway Intersections 14-66
14.10 Diverging Diamond Interchange (DDI) 14-67
14.10.1 Overview 14-67
14.10.2 Design Considerations 14-68
14.10.3 Sight Distance 14-71
14.10.4 Horizontal Alignment Alternatives 14-71
14.10.5 Auxiliary Lanes 14-73
14.10.6 Ramp Terminals 14-75
14.10.7 Pedestrian and Bicyclist Considerations 14-76
14.10.8 Access Management 14-78
14.10.9 Safety 14-78
14.10.10 Operations 14-79
14.10.11 Traffic Control Devices 14-79
14.10.12 Illumination 14-79
14.11 Contraflow Left Interchange (CLI) 14-80
14.12 Single-Point Urban Interchange 14-81
14.13 Roundabout Interchange 14-83
14.14 Partial Cloverleaf Interchange 14-84
Chapter 15 Grade Separations and Interchanges 15-1
15.1 Overview 15-1
15.2 Warrants for Interchanges and Grade Separations 15-3
15.3 Types of Interchanges 15-5
15.3.1 Three-Leg Interchanges 15-5
15.3.2 Four-Leg Interchanges 15-6
15.4 Applications on Various Type of Facilities 15-16
15.4.1 Urban Street Grade Separations 15-16
15.4.2 Multi-Lane Rural Highway Grade Separations 15-16
15.4.3 Freeway Grade Separations 15-16
15.5 Safety 15-17
15.5.1 Major Intersections with High Traffic Volumes 15-17

15.5.2 Railroad Crossings 15-17
15.5.3 Benefits 15-17
15.6 General Design Considerations 15-19
15.6.1 Interchange Spacing 15-19
15.6.2 Lane Balance 15-20
15.6.3 Auxiliary Lanes 15-20
15.6.4 Weaving Sections 15-20
15.6.5 Collector-Distributor Roads 15-20
15.7 Ramps & Direct Connectors 15-21
15.7.1 Types of Ramps 15-21
15.7.2 General Ramp Design Considerations 15-21
15.7.3 Ramp Geometrics 15-22
15.7.4 Cross Section and Cross Slopes 15-25
15.7.5 Lane and Shoulder Widths. 15-26
15.7.6 Sight Distance 15-27
15.7.7 Ramp Terminal Design 15-27
15.7.8 Ramp Spacing 15-36
15.7.9 Metered Ramps 15-36
15.8 Frontage Road Turnarounds and Intersection Approaches 15-38
15.8.1 Guidelines for Designing Auxiliary Lanes (AL) on Frontage Roads Approaching Intersections 15-41
Chapter 16 Driveways 16-1
16.1 Overview 16-1
16.1.1 Definitions 16-1
16.2 General Design Guidelines 16-4
16.2.1 Geometrics for Two-Way Driveways 16-6
16.2.2 Geometrics for Divided Driveways 16-10
16.2.3 Bicycle and Pedestrian Considerations 16-11
16.3 Profiles 16-12
16.3.1 Driveway Grades 16-12
16.3.2 Profiles on Curb and Gutter Sections 16-15
16.3.3 Profiles with Drainage Ditch 16-16
16.4 Driveway Angle 16-17
16.4.1 Two-Way Driveways 16-17
16.4.2 One-Way Driveways 16-17

16.5 Visibility 16-18
Chapter 17 Roadside Safety Design and Roadside Safety Hardware 17-1
17.1 Overview 17-1
17.2 Roadside Safety Hardware Crash Criteria 17-2
17.2.1 MASH Background 17-2
17.2.2 Current TxDOT MASH Implementation Timetable/Policy 17-3
17.3 Longitudinal Barriers 17-5
17.3.1 Types of Barriers 17-5
17.3.2 Applications 17-6
17.3.3 Working Width 17-9
17.4 Concrete Barriers (Median and Roadside) 17-10
17.4.1 Application 17-10
17.4.2 Location 17-10
17.4.3 Standard Installations 17-10
17.5 Guard Fence 17-11
17.5.1 Blockouts 17-11
17.5.2 Structural Considerations of Guard Fence 17-11
17.5.3 Placement of Guard Fence 17-13
17.5.4 End Treatment of Guard Fence 17-16
17.6 Determining Length of Need of Barrier 17-17
17.6.1 Variables 17-17
17.6.2 Design Equations 17-19
17.6.3 Using Design Equations to Determine Length of Guard Fence 17-20
17.6.4 Example Problems 17-21
17.7 Median Barrier 17-27
17.7.1 Application 17-27
17.7.2 Placement 17-28
17.8 Attenuators (Crash Cushions) 17-33
17.8.1 Application 17-33
17.8.2 Crash Cushion Categories 17-33
17.8.3 Sacrificial 17-33
17.8.4 Location 17-33
17.8.5 Standard Installations 17-34
17.9 Emergency Crossovers 17-35
17.9.1 Location 17-35

17.9.2 Construction 17-35
Chapter 18 Bicycle Facilities 18-1
18.1 General 18-1
18.1.1 Purpose 18-2
18.1.2 Micromobility and Power-Driven Mobility Devices on Bikeways 18-2
18.1.3 Definitions 18-3
18.1.4 Relationship to Other Policies, Laws, and Regulations 18-4
18.1.5 Projects that can be Exempt from Bicycle Accommodations 18-5
18.1.6 Use of Dimensional Values for Bikeway Design 18-7
18.1.7 Design Exceptions and Waivers for Dimensional Values of Bicycle Facilities 18-8
18.2 Planning and Context 18-10
18.2.1 Bicycle Planning Principles 18-10
18.2.2 Context Considerations 18-11
18.2.3 Target Design User 18-14
18.2.4 General Bikeway Selection 18-16
18.2.5 Bikeway Feasibility Assessment 18-20
18.3 Elements of Design 18-23
18.3.1 Design Characteristics of Bicyclists 18-23
18.3.2 Bicycles and Micromobility Devices 18-23
18.3.3 Bicycling Operating Space 18-24
18.3.4 Bicyclist Operating Speeds 18-25
18.3.5 Sight Distance 18-25
18.3.6 Horizontal Shifting Tapers 18-32
18.3.7 Surface Considerations 18-33
18.3.8 Utility Considerations 18-34
18.3.9 Drainage Considerations 18-34
18.3.10 Bikeway Curb Considerations 18-36
18.3.11 Railings and Barriers Adjacent to Bikeways 18-36
18.3.12 Intersection Elements 18-37
18.3.13 Bikeway Lighting 18-41
18.3.14 Restrict Motor Vehicle Use of Bicycle Facilities 18-42
18.4 Bikeway Types 18-44
18.4.1 Shared Use Paths Adjacent to Roadways (Sidepaths) 18-45
18.4.2 Separated Bike Lanes 18-55
18.4.3 Buffered Bike Lanes 18-61

18.4.4 Bike Lanes 18-63
18.4.5 Raised Bike Lanes 18-65
18.4.6 Bike Accessible Shoulders 18-68
18.4.7 Shared Lanes (wide outside lane) 18-72
18.4.8 Rural Bikeway Types 18-73
18.5 Intersections and Crossings 18-75
18.5.1 Principles of Intersection Design 18-76
18.5.2 Intersection Approach Treatments 18-78
18.5.3 Bicycle Ramps to Transition Between Bicycle Facilities 18-100
18.5.4 Driveways 18-103
18.5.5 Mid-Block Shared Use Path Crossings 18-104
18.5.6 Roundabouts 18-109
18.5.7 Railroad Crossings 18-113
18.6 Maintenance, Operations, and Work Zone 18-116
18.6.1 Maintenance Considerations 18-116
18.6.2 Temporary Traffic Control for Bicyclists / Maintenance of Traffic 18-117
Chapter 19 Pedestrian Facilities 19-1
19.1 Overview 19-1
19.1.1 Purpose 19-1
19.1.2 Design Imperative - Safety 19-2
19.1.3 Relationship to Other Policies, Laws, and Regulations 19-3
19.1.4 Definitions: 19-6
19.1.5 Provision of Pedestrian Facilities 19-7
19.1.6 Project Development 19-8
19.1.7 Maintenance and Operation 19-9
19.2 Elements of Design 19-10
19.2.1 Design User 19-10
19.2.2 Accessibility Requirements 19-11
19.3 Linear Pedestrian Facilities 19-17
19.3.1 Characteristics Contributing to Pedestrian Comfort and Safety Along the Roadway 19-17
19.3.2 Sidewalk Design on Curbed and Non-Curbed Roadways 19-17
19.3.3 Shared Use Paths 19-20
19.3.4 Linear Pedestrian Facility Design Criteria 19-21
19.4 Curb Ramp Design 19-23
19.4.1 Curb Ramp Locations 19-23

19.4.2 Design Considerations 19-23
19.4.3 Curb Ramp Components 19-24
19.4.4 Design Elements 19-24
19.4.5 Curb Ramp Types 19-28
19.4.6 Curb Ramp Evaluation 19-32
19.4.7 Design Variance to Curb Ramp Replacement 19-32
19.5 Driveway Design Considerations 19-34
19.5.1 General Guidelines 19-34
19.5.2 Designing for Cross Slope Compliance 19-36
19.6 Intersections and Crossings 19-40
19.6.1 Pedestrian Crossings Principles 19-40
19.6.2 Intersection Crossing Treatment Decision-Making Framework 19-41
19.6.3 Geometric Treatments for Pedestrians at Intersections 19-44
19.6.4 Uncontrolled Crossing Safety Countermeasures 19-50
19.6.5 Signalized Crossing Safety Countermeasures 19-56
19.6.6 Mid-block Pedestrian Crossing Safety Countermeasures 19-63
19.6.7 Pedestrian Facilities at Alternative Intersections 19-64
19.6.8 Pedestrian Treatments at Railroad Crossings 19-64
19.7 Overcrossings and Underpasses 19-66
19.7.1 Sidewalks for Bridges and Underpasses 19-66
19.7.2 Railings and Handrails for Pedestrian Facilities 19-67
19.7.3 Pedestrian/Bike Bridges and Underpasses 19-68
19.8 Work Zone and Temporary Traffic Control Pedestrian Accommodations 19-71
19.9 Lighting 19-73
19.10 On-Street Parking 19-75
19.10.1 Parallel Accessible Parking 19-75
19.10.2 Perpendicular Accessible Parking 19-76
19.10.3 Angled Accessible Parking 19-76
19.10.4 Curb Ramp 19-76
19.11 Railings Adjacent to Steep Slopes 19-78
19.12 Additional Considerations 19-80
19.12.1 Personal Security 19-80
19.12.2 Aesthetics 19-80
19.12.3 Exposure to the Elements 19-80
Chapter 20 Motorcyclist Design Considerations 20-1
20.1 Overview 20-1

20.2 Pavement Features 20-2
20.2.1 Metal Covers 20-2
20.2.2 High Friction Surfacing 20-2
20.2.3 Removal of Pavement Markings 20-3
20.2.4 Pavement Markings 20-3
20.3 Roadside Features 20-5
20.3.1 Vegetation 20-5
20.3.2 Signs and Other Road Furniture 20-6
20.3.3 Longitudinal Barriers 20-6
20.3.4 Shoulders 20-7
20.3.5 Clear Zones 20-7
20.4 Highway Features 20-9
20.4.1 Horizontal Curves (General) 20-9
20.4.2 Horizontal Curves (Specific) 20-9
20.4.3 Intersections 20-10
20.4.4 Roundabouts 20-11
20.4.5 Signs Related to Motorcyclists 20-11
20.5 Motorcycle Safety During Construction 20-13
20.5.1 Pavement 20-13
20.5.2 Construction Maintenance 20-13
20.6 Summary of Recommendations 20-14
Chapter 21 Texas Highway Freight Network, Texas Trunk System, National Highway System, STRAHNET and Hurricane Evacuation Routes 21-1
21.1 Overview 21-1
21.2 Texas Highway Freight Network (THFN) 21-2
21.2.1 Vertical Clearance at Structures 21-2
21.2.2 Signs, Overhead Sign Bridges (OSB's), Signals 21-3
21.2.3 Other Overhead Utilities 21-3
21.2.4 Bicycle and Pedestrian Accommodations 21-3
21.3 Texas Trunk System (TRUNK) 21-4
21.4 National Highway System (NHS) 21-5
21.4.1 History of the National Highway System 21-5
21.5 Strategic Highway Network (STRAHNET) 21-6
21.6 Hurricane Evacuation Routes 21-7
21.6.1 Hurricane Evacuation Lanes 21-7
21.6.2 Contraflow Lanes 21-7

Chapter 22 Transit 22-1
22.1 Overview 22-1
22.1.1 General Need 22-1
22.1.2 Types of Transit 22-2
22.1.3 Examples of Transit Agencies 22-2
22.1.4 Transit Mix 22-3
22.1.5 Transit Accommodation 22-3
22.2 Right of Way Requirements 22-4
22.2.1 Passenger Platforms 22-4
22.2.2 Intersections 22-4
22.2.3 Transit Centers 22-4
22.2.4 Dedicated Transitways 22-4
22.3 Roadway Design 22-5
22.3.1 Lane Widths 22-5
22.3.2 Profiles 22-5
22.3.3 Vertical Clearances 22-5
22.3.4 Acceleration and Deceleration 22-5
22.3.5 Transit Stops 22-6
Chapter 23 Temporary Traffic Control Design 23-1
23.1 Overview 23-1
23.2 General Standards and Practices 23-2
23.2.1 Purpose of a Traffic Control Plan (TCP) Design 23-2
23.2.2 Safety in TCP Design 23-2
23.2.3 Federal Regulations relevant to TCP Design 23-2
23.3 TCP Design Considerations 23-4
23.3.1 Duration 23-4
23.3.2 Cross Sectional Elements 23-4
23.3.3 Pedestrian and Bicycle Requirements 23-4
23.3.4 Property Access Requirements 23-7
23.3.5 Temporary Geometry Requirements 23-7
23.3.6 Temporary Pavement Structure Requirements 23-12
23.3.7 Temporary Drainage Requirements 23-12
23.3.8 Temporary Clearance Requirements 23-12
23.3.9 Temporary Ramp Design 23-14

23.3.10 Business Access Requirements 23-15
23.3.11 Emergency Vehicle Access Requirements 23-15
23.3.12 Treatment of Pavement Drop-offs in Work Zones 23-15
Chapter 24 Other Design Elements 24-1
24.1 Overview 24-1
24.2 Fencing 24-2
24.2.1 Overview 24-2
24.2.2 Property Fencing on ROW 24-2
24.2.3 Control of Access Fencing on Freeway 24-2
24.3 Illumination 24-4
24.3.1 Overview 24-4
24.3.2 General Considerations 24-4
24.3.3 Other Safety Benefits 24-5
24.4 Signing 24-6
24.4.1 Overview 24-6
24.4.2 General Considerations 24-6
24.4.3 Other Safety Benefits 24-6
24.5 Pavement Markings 24-7
24.5.1 Overview 24-7
24.5.2 General Considerations 24-7
24.5.3 Other Safety Benefits 24-7
24.6 Intelligent Transportation Systems (ITS) 24-8
24.6.1 Overview 24-8
24.6.2 General Considerations 24-8
24.6.3 Other Safety Benefits 24-8
24.7 Wildlife Crossings 24-9
24.7.1 Overview 24-9
24.7.2 Background 24-9
24.7.3 Overpass Crossings 24-9
24.7.4 Underpass Crossings 24-10
24.7.5 Culvert Crossings 24-10
24.7.6 Barriers 24-11
24.7.7 Wildlife Guards 24-11
24.7.8 Fencing and Signing 24-12
24.8 Texas Aggregate Quarry and Pit Safety Act 24-13

Appendix A Minimum Design Criteria for Projects Requiring a Design Exception A-1
Appendix B Minimum Design Criteria for Projects Requiring a Design Waiver B-1
Appendix C List of References C-1

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
General Guidance

1.1 Overview

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

The guidelines may require modification for local conditions to cover all highway design situations. It is important that significant deviations from the manual be documented and be based on an objective engineering analysis.

Roadway design criteria and technology are rapidly changing. New design values that 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 intent 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 considering environmental quality.

1.1.1 Project Objectives and Goals

Each project should begin with an explicit statement that indicates why the project is being undertaken and what the project is intended to accomplish. This statement should be documented and can be referred to in the **Design Summary Report (DSR).**

The Project Scoping portion of the DSR should contain information including what will, and what will not, be undertaken in the project. This includes the project problems and proposed solutions, and project goals and objectives. The Project Scope should be built around an assessment of past performance of all transportation modes and a forecast of future performance. It should identify aspects of performance and metrics for performance improvement.

The Project Scoping Statement will be documented in the DSR.

1.1.2 Framework for Geometric Design

Historically, a range of "minimum" to "desirable" criteria for design values were provided with the "desirable" design criteria preferred. **American Association of State Highway and Transportation Officials (AASHTO) A Policy on Geometric Design of Highways and Streets** provides a framework by which projects should be assessed to determine appropriate design values rather than defaulting to a "minimum" or "desirable" value. This value is shown as the **"target" value** throughout the remainder of the manual.

"Target" values are meant to serve as a starting point for selecting the specific design criteria. The final selected design criteria value may be more or in some cases less (a "minimum" value) than the "target" value. The designer should use their engineering judgement to select the final design criteria based on the functional classification, context classification and performance metrics of the project established during project scoping.

Similarly, the RDM provides the following framework based on five factors:

- **Functional Classification** – characterizes roadways by their position in the transportation network and the type of service it provides for vehicular traffic. See **Chapter 3.**
- **Context Classification System** – characterizes roadways by their surrounding environment and their place in the community. See **Chapter 3.**
- **Modal Classification System** – considers the needs of all transportation modes in design of the roadway. See **Chapter 3.**
- **Project Type of Construction** – incorporates a design process to choose the appropriate design approach for each project. See **Section 1.1.3.**
- **Practical Design** – an approach to finding and balancing the needs of all users for each project. See **Chapter 2.**
- **Performance-based Design** – an approach for considering the effects of geometric design decisions. See **Chapter 2.**

The concept of **Performance-Based Practical Design (PBPD)** is a decision-making approach that helps agencies better manage transportation investments and serve system-level needs and performance priorities with limited resources. See **Chapter 2** for a full discussion of PBPD.

1.1.3 Types of Construction

The RDM describes design criteria for the following categories of highway construction: resurfacing, restoration, rehabilitation, and reconstruction.

Table 1-1 gives definitions of the different highway construction categories.

Table 1-1: Highway Construction Category Definitions

Highway Construction Category	Definition
Resurfacing	Limited existing pavement surface repairs such as milling, spot patching, seal coat, rut repair, crack sealing, and fog sealing.
Restoration	Existing pavement structure repairs such as patching or rebuilding short sections of road, milling and hot mix overlays, shoulder edge repairs, or minor safety improvements (i.e., guardfence, pipe headwalls, etc.).
Rehabilitation	Partial or full depth pavement reconstruction on existing profile/alignment such as removal and replacement of the existing riding surface and base material, driveway improvements, ditch regrading or safety improvements (i.e., guardfence, pipe headwalls, etc.). If approved, minor lane/shoulder widening, adding medians/curbing, minor improved drainage design or intersection improvements are included.
Reconstruction	Projects that utilize an existing roadway alignment (or make only minor changes to an existing alignment) but involve a change in the basic roadway type. Changes in the basic roadway type include widening a road to provide additional through lanes or adding a raised or depressed median where none currently exists, and where these changes cannot be accomplished within the existing roadway width (including shoulders).
New Construction	Projects on a new alignment or new construction.

TxDOT designates the following as applicable design criteria in **Table 1-2** using the definitions above.

Table 1-2: TxDOT Construction Types and Applicable Design Criteria

Roadway Design Criteria	Highway Construction Categories	Definition
2R	Resurfacing and Restoration	Projects consist of non-freeway work on facilities not on the National Highway System (NHS) with a current ADT ≤ 2500 per lane that propose to restore the pavement to its original condition. ○ Includes restoration of pavement structure, ride quality, or other necessary components to its existing cross section configuration. The principal purposes of these projects are surfacing and repair of the pavement structure. ○ 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. ○ Work **DOES NOT** include the addition of through travel lanes.
3R	Resurfacing, Restoration and Rehabilitation	• Projects consist of transportation projects that extend the service life and enhance the safety of a roadway including freeways and NHS roadways. • In addition to the work described under resurfacing and restoration, the activities include upgrading the geometric design and safety of the facility. Work may include the upgrading of geometric features such as roadway widening, minor horizontal and vertical realignment (typically less than 10% of the project length) and improving bridges to meet current standards for structural loading and to accommodate the approach roadway width. • Work **DOES NOT** include the addition of through travel lanes.
4R	Resurfacing, Restoration, Rehabilitation, and Reconstruction and New Construction	• Projects on a new alignment or new construction. • Projects that upgrade an existing roadway to meet geometric criteria for a new facility. • Projects that include substantial changes in the geometric character of the highway, such as widening to provide additional through lanes, significant horizontal or vertical realignments, and major improvements to the pavement structure to improve long term service. • Bridge replacement projects with ADT > 400.
5R	Mobility Corridors	Projects that are intended to regenerate, or produce new, long-term transportation opportunities including multiple modes such as rail, utilities, freight, and passenger characteristics.
Notes:		
Guidance on Preventative Maintenance (PM) projects can be found in TxDOT's **Maintenance Management Manual** and the **Maintenance Operations Manual**.		

1.1.4 Roadway Design Manual Format

The RDM is formatted to present general guidance, concepts of design presented in **AASHTO's A Policy on Geometric Design of Highways and Streets** (i.e., the Green Book) and basic design criteria. **Table 1-3** lists the chapters and chapter descriptions of the RDM.

Table 1-3: RDM Chapter Contents

Chapter	Chapter Name	Description
1	General Guidance	Describes the design elements that require Design Exceptions, Design Waivers, Design Variances, and the Texas Highway Freight Network's Design Deviations. Also includes references for schematic design, access control, preliminary engineering, and maintenance in design.
2	Performance-Based Practical Design Concepts	Discusses the concepts and application of performance-based practical design.
3	Context and Facility Type Considerations	Describes the five context classifications along with considerations for all modes of users and traffic networks.
4	Basic Design Criteria	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.
5	Local Roads(4R)	Describes target design criteria for Local Roads.
6	Collectors(4R)	Describes target design criteria for Collector roads and streets.
7	Arterials (4R)	Describes target design criteria for Arterial roads and streets.
8	Freeways (4R)	Describes target design criteria for freeways. Freeways include Interstates, Expressways, and Freeways.
9	Mobility Corridor Facilities (5R)	Describes and provides design guidance on mobility corridors with design speeds of 85 mph to 100 mph.
10	3R Design Criteria	Describes 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.
11	2R Design Criteria	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.
12	Special Facilities	Describes special facility design criteria. Special facilities may include off-system bridge projects, historical roadways or structures, and park roads. 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.
13	Intersections	Discusses types of intersections and design criteria and considerations for the design of intersections.
14	Alternative Intersections and Interchanges	Discuss types of alternative intersections and interchanges and design criteria and considerations for the design of alternative intersections and interchanges.

Chapter	Chapter Name	Description
15	Grade Separations and Interchanges	Discusses grade separations and interchanges. Describes design criteria for ramps.
16	Driveways	Discusses design criteria and considerations for the design of driveways.
17	Roadside Safety Design and Roadside Safety Hardware	Discusses the design of roadside safety and roadside safety hardware.
18	Bicycle Facilities	Describes design criteria and considerations for the design of bicycle facilities
19	Pedestrian Facilities	Describes design criteria and considerations for the design of pedestrian facilities.
20	Motorcyclist Design Considerations	Discusses motorcyclist design considerations.
21	Texas Highway Freight Network, Texas Trunk System, NHS, STRHNET, and Hurricane Evacuation Routes	Describes the policy for highways on the Texas Highway Freight Network.
22	Transit	Discusses the design of transit for buses and trains as it pertains to vehicular roadways.
23	Temporary Traffic Control Design	Discusses design criteria and considerations for the design of temporary traffic control.
24	Other Design Elements	Discusses roadway design considerations for fencing, illumination, signing, pavement markings, intelligent transportation systems (ITS),wildlife crossings, and the Texas Aggregate Quarry and Pit Safety Act
Appendix A	Minimum Values for Projects Requiring a Design Exception	Tables of the minimum design values, that if not met, would require a design exception as described in Chapter 1.
Appendix B	Minimum Values for Projects Requiring a Design Waiver	Tables of the minimum design values, that if not met, would require a design waiver as described in Chapter 1.
Appendix C	List of References	List of references found in the manual with links.

1.2 Design Exceptions, Design Waivers, Design Variances, Texas Highway Freeway Network Design Deviations

1.2.1 Design Exceptions

A design exception is required when the minimum values of the controlling criteria specified in this Chapter for the respective categories of construction projects are not met. A design exception is not required when values meet or exceed the minimum guidelines.

The controlling criteria for the following project categories will require a **design exception.**

Minimum values requiring a design exception for each of the controlling design criteria can be found in Appendix A.

1.2.1.1 New Location and Reconstruction Projects (4R)

The adopted Federal Highway Administration (FHWA) design requirements in 23 CFR 625, and 49 CFR 37.9 apply to new location and reconstruction projects (4R) on the National Highway System (NHS).

Table 1-4 shows the list of AASHTO's ten controlling design criteria that will require a **design exception** when the minimum requirements specified in Appendix A are not met.

A design exception **is not** required when values meet or exceed the minimum requirements.

Table 1-4: AASHTO's Controlling Criteria for New Location and Reconstruction Projects (4R)

Criteria	Minimum Criteria	Criteria Reference	Explanation
Design Speed	Appendix Table A-1	Section 4.2	Minimum design speed based on the functional classification.
Lane Width	Appendix Table A-2	Note 1	Minimum lane width based on functional classification, type of lane (Mainlane, etc.) and design speed.
Shoulder Width	Appendix Table A-3	Note 2	Minimum outside and inside shoulder widths based on functional classification, type of facility and design speed.
Horizontal Curve Radius	Appendix Table A-4	Section 4.7.2	Minimum horizontal curve radius based on design speed and maximum superelevation rate.
Superelevation Rate	Table 4-4 through Table 4-7	Section 4.7.3	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.

Criteria	Minimum Criteria	Criteria Reference	Explanation
Stopping Sight Distance (SSD)[3]	Appendix Table A-5	Section 4.11.1	The minimum distance a driver 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	Appendix Table A-6	Section 4.8.1	Maximum gradient of a roadway along the centerline. For maximum grade for ramps see Table 15-2.
Cross Slope	Appendix Table A-7	Section 4.10.4	Maximum transverse slope across a pavement surface, excluding superelevation.
Vertical Clearance	Appendix Table A-8	Section 4.8.6	Minimum specified height of a bridge or overhead projection above the roadway.
Design Loading Structural Capacity	Appendix Table A-9	TxDOT Bridge Design Manual - LRFD	The Design Load Structural Capacity-Maximum Load that a bridge is designed to handle.

Notes:

1. For Local Roads see Section 5.1.6; for Collectors see Section 6.1.6; for Arterials see Section 7.1.6; for Freeways see Sections 8.1.6; for Mobility Corridors see Section 9.2.2.
2. For Local Roads see Section 5.1.7; for Collectors see Section 6.1.7; for Arterials see Section 7.1.7; for Freeways see Sections 8.1.7; for Mobility Corridors see Section 9.2.3.
3. 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 (Figure 4-6), then the vertical SSD is satisfied under usual conditions.

TxDOT implements additional controlling criteria for design elements that will require a **design exception** if the conditions are not met on new location and reconstruction projects (4R).

Table 1-5 shows the list of TxDOT's additional controlling criteria for new location and reconstruction projects that will require a **design exception** when the minimum requirements are not met.

A design exception **is not** required when values meet or exceed the minimum requirements.

Table 1-5: TxDOT's Controlling Criteria for New Location and Reconstruction Projects (4R)

Criteria	Minimum Criteria	Criteria Reference	Explanation
Bridge Class Culvert Protection	Appendix Table A-10	TxDOT Bridge Railing Manual	All ADTs
Bridge Rail	Appendix Table A-11	TxDOT Bridge Railing Manual	Bridge rail must comply with MASH and meet requirements of the TxDOT Bridge Railing Manual.

1.2.1.2 Resurfacing, Restoration or Rehabilitation Projects (3R)

TxDOT implements **controlling criteria** for the following design elements that will require a **design exception** if the minimum conditions are not met on resurfacing, restoration, or rehabilitation projects (3R).

Table 1-6 shows the list of TxDOT's additional controlling criteria that will require a **design exception** when the minimum requirements are not met.

A design exception **is not** required when values meet or exceed the minimum requirements.

For 3R projects, "high-volume" roadways are those with greater than or equal to a current ADT of 1,500.

"Low-volume" roadways are defined as those with a current ADT of less than 1,500.

Table 1-6: TxDOT's Controlling Criteria for Resurfacing, Restoration or Rehabilitation Projects (3R)

Criteria	High Volume ≥ 1500 ADT	Low Volume < 1500 ADT	Minimum Criteria	Criteria Reference	Explanation
Design Speed	X		Appendix Table A-12	Section 10.1.5.2	Minimum design speed based on the highway class and context.
Lane Width	X	X	Appendix Table A-13	Section 10.1.5.4	Minimum lane width based on the respective functional classification, context classification, and current Average Daily Traffic.
Shoulder Width	X	X	Appendix Table A-14	Section 10.1.5.4	Minimum shoulder width based on the respective functional classification, context classification, and current Average Daily Traffic.
Horizontal Curve Radius	X		Appendix Table A-15	Section 10.1.5.1	Minimum horizontal curve radius based on design speed and maximum superelevation rate.
Superelevation Rate	X		Table 4-4 through Table 4-7	Section 10.1.5.1	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)[3]	X		Appendix Table A-16	Section 4.11.1	The minimum distance a vehicle needs to be able to see to have room to stop before colliding with something in the roadway. Use K-values to assist calculation.
Maximum Grade	X		Appendix Table A-17	Section 4.8.1	Maximum gradient of a roadway along the centerline.
Design Loading Structural Capacity	X	X	Appendix Table A-18	TxDOT Bridge Design Manual - LRFD	The Design Load Structural Capacity-Maximum Load that a bridge is designed to handle.
Bridge Class Culvert Protection	X		Appendix Table A-19	TxDOT Bridge Railing Manual	All ADTs
Deficient Bridge Rail	X		Appendix Table A-20	TxDOT Bridge Railing Manual	Minimum requirements are met for new bridge rail or existing rail to remain in place.

Notes:

1. SSD applies to horizontal alignments and 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 4-6), then the vertical SSD is satisfied under usual conditions.

1.2.1.3 Resurfacing or Restoration Projects (2R)

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

A design exception **is not** required when values meet or exceed the minimum requirements.

1.2.1.4 Bicycle Facilities

Table 1-7 gives the controlling criteria for bicycle facility projects that will require **a design exception** when the minimum requirements are not met.

A design exception **is not** required when values meet or exceed the minimum requirements.

See **Section 18.1.7** for additional information on design exceptions for bicycle facilities.

Table 1-7: Controlling Criteria for Bicycle Facilities

Criteria	Minimum Criteria	Criteria Reference	Explanation
Bike Lane Width	Appendix Table A-21	Section 18.4.4.2	Minimum bike lane width requirement based on context classification.
Bike Shared Lane Width (Wide Outside Lane)	Appendix Table A-22	Section 18.4.7.2	Minimum lane width requirement based on context classification.
Bridge Deck Clear Space	Appendix Table A-23	Note 1	Minimum shoulder and offset measured to the toe of barrier provided on the structure.
Notes:			
1. For Local Roads see Section 5.1.7; for Collectors see Section 6.1.7; for Arterials see Section 7.1.7.			

1.2.1.5 Special Facilities

Table 1-8 gives the requirements for off-system bridge replacement and rehabilitation projects with current ADT of 400 or less that will require a **design exception** when the minimum requirements are not met.

A design exception **is not** required when values meet or exceed the minimum requirements.

If the current ADT is greater than 400, then applicable 4R or 3R criteria applies.

Table 1-8: TxDOT's Controlling Criteria for Off-System Bridges with ≤ 400 ADT

Criteria	Minimum Criteria	Criteria Reference	Explanation
Design Speed	Appendix Table A-24	Section 12.2.2	Minimum design speed based on condition of the remainder of the road.
Stopping Sight Distance (SSD)[1]			Minimum SSD to be attained based on the condition of the remainder of the road.
Horizontal Curve Radius			Minimum horizontal radius based on design speed and the condition of the remainder of the road.
Superelevation Rate			Minimum superelevation to be used based on the condition of the remainder of the road
Maximum Grades			Maximum grade to be used based on the condition of the remainder of the road.
Minimum Structure Width			Minimum width measured between face to face of rail depending on approach roadway width.
Bridge End Guard Fence			Minimum length of transition and end treatment use.
Approach Roadway Width			Minimum approach width, transition, and surfacing.
Design Loading Structural Capacity			The Design Load Structural Capacity-Maximum Load that a bridge is designed to handle.
Traffic Control			Traffic control for detours and placement of traffic control devices.
Notes:			
1. Stopping Sight Distance (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 (Figure 4-6), then the vertical SSD is satisfied under usual conditions.			

1.2.1.6 Off-System Historically Significant Bridge Projects

Table 1-9 gives the controlling criteria for off-system historically significant bridge projects that will require a **design exception** when the minimum requirements are not met.

A design exception **is not** required when values meet or exceed the minimum requirements.

Table 1-9: TxDOT's Controlling Criteria for Off-System Historically Significant Bridges

Criteria	ADT	Minimum Criteria	Criteria Reference	Explanation
Roadway Width	≤ 400	Appendix Table A-25	Section 12.3	Minimum roadway width based on the condition of the remainder of the road.
Load Carrying Capacity (Operating Rating)				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.

1.2.1.7 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 (TPWD).

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.

A design exception or design waiver **is not** required when values meet or exceed the minimum requirements.

A Design Exception form and other information can be found on the Design Division's website (TxDOT intranet only).

1.2.2 Design Waivers

When criteria are not met for non-controlling design elements, a design exception is not required. However, a departure from the minimum criteria outlined in the RDM will be handled by **design waivers** at the District level. Design waivers will be granted as the District authorizes. A design waiver is not required when values meet or exceed the minimum requirements.

TxDOT's non-controlling criteria for the following project categories will require a **design waiver.**

Minimum values requiring a design waiver for each of the non-controlling design criteria can be found in Appendix B.

1.2.2.1 New Location and Reconstruction Projects (4R)

Table 1-10 gives TxDOT's non-controlling criteria for 4R projects that will require a **design waiver** when the minimum requirements are not met. A design waiver **is not** required when values meet or exceed the minimum requirements.

Table 1-10: TxDOT Non-Controlling Criteria for 4R Projects

Criteria	Minimum Criteria	Criteria Reference	Explanation
Curb Parking Lane Width	Appendix Table B-1	Section 4.10.16	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	Appendix Table B-2	Section 4.10.2 and 8.1.8	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	Appendix Table B-3	Section 4.10.2	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.
Curb Offset	Appendix Table B-4	Section 4.10.12	Minimum distance between the edge of the travel lane and the face of the curb.
Bridge Width	Appendix Table B-5	TxDOT Bridge Project Development Manual	Minimum overall bridge width.
Median Width	Appendix Table B-6	Section 4.10.15	Minimum width of separation between dual roadways which separates the traffic flow in the opposite direction.
Clear Zone	Appendix Table B-7	Section 4.10.7	The minimum unobstructed, traversable area provided beyond the edge of the through traveled way for the recovery of errant vehicles.
Lateral Offset to Obstructions	Appendix Table B-8	Section 4.10.8	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	Appendix Table B-9	TxDOT Bridge Project Development Manual	Minimum vertical and horizontal clearances.
Sag Vertical Curve Length[1]	Appendix Table B-10	Section 4.8.2	Length of sag vertical curve.
Superelevation (Non-rate elements)	Appendix Table B-11	Sections 4.7.6 and 4.7.7	Non-rate elements include length and location of superelevation transitions.
Guardrail Length	Appendix Table B-12	Section 17.3	The Length of Need (LON) of guardrail required to protect both adjacent and opposing traffic, except where access accommodation is needed.
Shared Use Path Width	Appendix Table B-13	Section 18.4.1	Minimum width for Bikeways that are separated from vehicular traffic that also allow pedestrians and non-motorized users.

Criteria	Minimum Criteria	Criteria Reference	Explanation
Separated Bike Lane/ Buffered Bike Lane Width	Appendix Table B-13	Sections 18.4.2 and 18.4.3	Minimum width for Bikeways that are separated from vehicular traffic by vertical elements or striped buffer areas in the street.
Bike Accessible Shoulder Width	Appendix Table B-13	Section 18.4.6	Minimum width for Bikeways on a roadway adjacent to vehicle traffic.

Notes:

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 4-7), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings, see Figure 4-29. See Section 4.8.2 for additional guidance on comfort criteria.

1.2.2.2 Resurfacing, Restoration or Rehabilitation Projects (3R)

Table 1-11 gives TXDOT's non-controlling criteria for 3R projects that will require a **design waiver** when the minimum requirements are not met.

A design waiver **is not** required when values meet or exceed the minimum requirements.

Table 1-11: TxDOT's Non-Controlling Criteria for Resurfacing, Restoration or Rehabilitation Projects (3R)

Criteria	High Volume ≥ 1500 ADT	Low Volume < 1500 ADT	Minimum Criteria	Criteria Reference	Explanation
Design Speed		X	Appendix Table B-14	Section 10.1.5.2	Minimum design speed based on the functional classification.
Horizontal Curve Radius		X	Appendix Table B-15	Section 10.1.5.1	Minimum horizontal curve radius based on design speed and maximum superelevation rate.
Superelevation Rate		X	Table 4-4 through Table 4-7	Section 10.1.5.1	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	Appendix Table B-16	Section 4.11.1	Minimum SSD to be attained based on a vehicle's ability to stop before reaching a stationary object in its path, at a certain design speed.
Maximum Grade		X	Appendix Table B-17	Section 4.8.1	Maximum gradient of a roadway along the centerline.

Criteria	High Volume ≥ 1500 ADT	Low Volume < 1500 ADT	Minimum Criteria	Criteria Reference	Explanation
Superelevation (Non-rate elements)	X	X	Appendix Table B-18	Sections 4.7.6 and 4.7.7	Non-rate elements include length and location of superelevation transitions
Bridge Width	X	X	Appendix Table B-19	TxDOT Bridge Project Development Manual	Minimum overall bridge width.
Deficient Bridge Rail		X	Appendix Table B-20	TxDOT Bridge Railing Manual	Minimum requirements are met for new bridge rail or existing rail to remain in place.
Clear Zone	X	X	Appendix Table B-21	Section 4.10.7	The minimum unobstructed, traversable area provided beyond the edge of the through traveled way for the recovery of errant vehicles.
Speed Change Lane Width	X	X	Appendix Table B-22	Section 4.10.2	Minimum width of acceleration or deceleration lanes for left or right turns, exits or entrance lanes, or a climbing lane based on the highway class.
Length of Speed Change Lanes	X	X	Appendix Table B-23	Section 4.10.2	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.
Curbside Parking Lane Width	X	X	Appendix Table B-24	Section 4.10.16	Minimum width of parallel parking lanes based on the functional class of the roadway.
Guardrail Length	X	X	Appendix Table B-25	Section 17.6	The Length of Need (LON) of guardrail required to protect adjacent and opposing traffic, except where access accommodation is needed.
Sag Vertical Curve Length[2]	X	X	Appendix Table B-26	Section 4.8.2	Length of sag vertical curve.

Notes:

1. SSD only applies to crest vertical curves for the purposes of a Design Waiver for 3R low-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 4-6), 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 4-7), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings, see Figure 4-29. See Section 4.8.2 for additional guidance on comfort criteria.

1.2.2.3 Resurfacing or Restoration Projects (2R)

Design waivers are not applicable to 2R projects.

1.2.2.4 Bicycle Facilities

Table 1-12 gives TxDOT's non-controlling criteria for bicycle facility projects that will require a **design waiver** when the minimum requirements given in **Chapter 18** cannot be met.

A design waiver **is not** required when values meet or exceed the minimum requirements.

Table 1-12: TxDOT's Non-Controlling Criteria for Bicycle Facilities

Criteria	Minimum Criteria	Criteria Reference	Explanation
Shared Use Path Width	Appendix Table B-13	Section 18.4.1	Minimum requirements based on context classification.
Separated Bike Lane/Buffered Bike Lane Width	Appendix Table B-13	Sections 18.4.2 and 18.4.3	Minimum requirements based on context classification.
Bike Accessible Shoulder Width	Appendix Table B-13	Section 18.4.6	Minimum requirements based on context classification.

1.2.2.5 Special Facilities

Table 1-13 gives TxDOT's non-controlling criteria for off-system bridge replacement and rehabilitation projects with current ADT of 400 or less that will require a **design waiver** when the minimum requirements are not met.

A design waiver **is not** required when values meet the minimum requirements.

Table 1-13: TxDOT's Non-Controlling Criteria for Off-System Bridges with ≤ 400 ADT

Criteria	Minimum Criteria	Criteria Reference	Explanation
Superelevation (Non-rate elements)	Appendix Table B-11	Section 12.2.2	Non-rate elements include length and location of superelevation transitions.
Sag Vertical Curve Length[1]	Appendix Table B-10	Section 12.2.2	Length of sag vertical curve.
Bridge Width	Appendix Table B-5	Section 12.2.2	Minimum overall bridge width.

Notes:

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 4-7), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings, see Figure 4-29. See Section 4.8.2 for additional guidance on comfort criteria.

1.2.2.6 Off-System Historically Significant Bridge Projects

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

1.2.2.7 Park Road Projects

Design waivers are not applicable to off-system park road projects. On-system Park Road projects need to meet the applicable criteria.

A Design Waiver form and other information can be found on the Design Division's website (TxDOT intranet only).

1.2.3 Design Variances for Pedestrian Facilities

A **design variance** is required to be sent to the Texas Department of Licensing and Regulation (TDLR) whenever the design guidelines specified in the **Texas Accessibility Standards (TAS)** and **Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-way (PROWAG)** are not met. Design variances should be sent to the Design Division – Landscape Architecture Branch for review prior to being submitted to TDLR for approval. See Chapter 19 for additional discussion on pedestrian facilities. This documentation should be kept in the project files.

A Design Variance form and other information can be found on the Design Division's website (Landscape Architecture Branch) (TxDOT intranet only).

1.2.4 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 **Sections 4.8.6** and **21.2** for applicable project criteria, and **Table 4-13** for vertical clearance values. **The Deviation request is exclusive of (and if applicable in addition to) the design exception that may be needed for vertical clearances not meeting the specified requirements in Section 4.8.6.** of the respective roadway facility.

The THFN Deviation requests must be submitted to the Design Division (DES) – Project Delivery Section and will be reviewed by an assigned Design Deviation Committee.

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

1.2.5 Approval Process

1.2.5.1 Design Exceptions

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 constitutes a quorum for recommending signature action. Since the construction plans

are sealed, the design exception documentation does not require an engineer's seal.

As per the current **Stewardship and Oversight Agreement on Project Assumption and Program Oversight by and Between Federal Highway Administration, Texas Division Office, and the Texas Department of Transportation (TxDOT)**, delegation of review and approval of design exceptions on the Interstate Highway System has been assigned to TxDOT. 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 Stewardship and Oversight (S&O) plan identifies specific design exception approval responsibility to FHWA, then design exceptions proposed for that project will be required to be submitted to FHWA for 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 PS&E Transmittal Data Form 1002.

Table 1-14 lists the Design Exception Approval Authority. The final signed Form 1002 all design exception forms and related documentation should be filed in ProjectWise and permanently retained.

For design exceptions on Interstates, refer to the **Design Exception Request for Interstate Highways TxDOT Standard Operating Procedures** found on the DES-PDS webpage.

Table 1-14: Design Exception Required Approvals

Project Type	Approval Authority	Submit Form To
State oversight projects	TxDOT District Engineer	District TP&D
Federal oversight projects	TxDOT District Engineer	District TP&D
Interstate projects[1]	TxDOT DES	DES – Project Delivery Section
"Involved projects" (TxDIP)	Based on TxDIP S&O plan	DES – Project Delivery Section
Design Loading Structural Capacity or Bridge Railing, or Bridge Class Culvert protection	Bridge Division (BRG)	BRG - Design Section

Notes:

1. Vertical clearance of < 16-ft on Interstate highways requires additional coordination and approval from the Department of Defense.

1.2.5.2 Design Waivers and ADA Design Variances

1.2.5.2.1 Design Waivers

Design waivers will be granted as the District authorizes. Complete documentation should be retained permanently in the District project files.

1.2.5.2.2 ADA Design Variance

An Americans with Disabilities Act (ADA) Design Variance form, process flowchart, and other information can be found on the **Design Division's (Landscape Architecture Branch) website.**

1.2.5.3 Texas Highway Freight Network (THFN) Design Deviation

A THFN Design Deviation form, process flowchart, and other information can be found on the Design Division's website (Project Delivery Section).

Table 1-15 lists the approval authority for design waivers, design variances, and design deviations.

Table 1-15: Approval Authority for Design Waivers, Design Variances, and Design Deviations

Type	Approval Authority	Submit Form To
Design Waiver	TxDOT District	District TP&D
ADA Design Variance	TDLR	DES – Landscape Architecture Branch
THFN Design Deviation	DES	DES – Project Delivery Section

1.3 Schematic Development

1.3.1 Schematic Development Process

Refer to **TxDOT's Project Development Process Manual (PDP)** for a comprehensive discussion of the schematic development process.

1.3.2 Types of Schematics

The types of schematics that are discussed in detail in the PDP include:

- **Geometric Schematic** – a geometric schematic that goes through a formal design, review and approval process is required for projects that are a:
    - New location project;
    - Added capacity project (including frontage roads);
    - Reconstruction with added ROW;
    - Interstate ramp relocation project;
    - Environmental Impact Statement project; or
    - Other projects that would require an IAJR.

    For high profile and politically sensitive projects, a geometric schematic is also recommended but not required.

    Refer to **Chapter 4 of the PDP** for more information about the Geometric Schematic.

- **Geometric Layout** – for projects that do not require the formal geometric schematic design, review, and approval process. This is a more simplified layout to convey project information and is used to gain feedback from project stakeholders. Refer to **Chapter 4 of the PDP** for more information.

1.3.3 Geometric Schematic Approval Process

Refer to **Chapter 4 of the PDP** for details concerning the submission, review and approval of schematics.

1.4 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. **Frontage roads are not considered part of the Interstate System.** 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 USC 103(c)(4)(A)** or other legislation.
- Routes approved as a future part of the Interstate system under **23 USC 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 ROW 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 **Interstate Access Justification Report Engineering, Operations and Safety Analysis TxDOT Standard Operating Procedures (IAJR SOP)** for further guidance on the development of Interstate Access Justification Reports and submittal process through the DES Division. A copy of the current TxDOT IAJR policy is available on the DES Division webpage and TxDOT.gov.

Refer to TxDOT's **Access Management Manual (AMM)** for a comprehensive discussion aspects of access criteria.

1.5 Preliminary Design

As outlined in the **PDP**, 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 RDM and other applicable guidelines and policies. Refer to the **PDP** for more discussion on preliminary design activities and requirements.

1.6 Maintenance Considerations in Design

The future maintenance of a facility cannot be overemphasized in project design. Reduced or low maintenance project designs with limited worker exposure should be an important 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 or interlocal agreement when necessary to grade outfalls and thus provide adequate drainage.
- Avoid designing drainage outfall that is lower than the adjacent property elevation.
- Design drainage structures with preferred velocities of 3-ft/s or greater to provide self-cleaning if possible.
- Reinforced concrete boxes (RCBs) should be no less than 3-ft tall to facilitate ease of cleanout and maintenance unless the roadway profile is severely constrained.
- Where practical, try to match the drainage structure to the natural grade of the drainage channel, and then profile the roadway over the structure.
- If existing conditions have silting or erosion, provide grades back to existing conditions and add protection.
- Avoid placing signs or impeding structures in ditches. 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.
- 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. Locate roadway infrastructure objects such that they are least likely to be struck by an errant vehicle.
- When designing grade separations, consider extending riprap on the header banks of the overpasses all the way to the crossroad pavement. Eliminate the need to mow or maintain small strips of soil or vegetation near objects, walls, roads, gores, etc.
- 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, roadway designs should reduce the amount of hand trimming or labor 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.
- Use desirable design criteria recommended herein regarding maximum roadway side slope ratios and ditch profile grades to the extent possible to reduce the maintenance effort and make required maintenance operations easier and safer to accomplish.

Chapter 2
Performance-Based Practical Design Concepts

2.1 Overview

The concepts of Performance-Based Practical Design (PBPD) date from the early 2000s. Their formation and development have been motivated by two main recognitions:

- The road building industry must do business in a more financially sustainable, results-oriented, and context sensitive way.
- A more flexible and data-driven design approach is necessary to realize this objective.

These recognitions have prompted the development of data-based tools and compilation of data to determine the associations between roadway features and actual roadway performance.

PBPD is a decision-making approach that helps agencies better manage transportation investments and serve system-level needs and performance priorities with limited resources.

This leads to a greater understanding of what improvements or physical features are likely to produce desired outcomes and allows design decisions to be made based on defensible and reproducible analyses.

Research that has analyzed safety and other data has found that some long-standing practices and long-held assumptions did not necessarily reflect reality.

Even for elements known to have sizable effects on performance, the criteria that govern their selection are often overly simplistic and insensitive to the full range of context, leading to their misuse (e.g., selection of larger horizontal curve radii or "desirable" lane and shoulder widths without thought to how the performance is affected). **Rigid adherence to dimensional guidance without understanding the nuance of how small variations affect performance leads to large expenditures with little benefit.** An increased knowledge base allows for a more flexible, confident, and cost-effective design approach.

Through PBPD, TxDOT can weigh project-level results and associated trade-offs against system-wide performance needs and goals. By focusing on system-wide performance, TxDOT can better manage the cumulative effectiveness of individual project investments and build upon the goals of Context Sensitive Solutions, Flexibility in Design, Practical Design, Asset Management, and Value Engineering. A PBPD approach relies on **quantitative analyses** to guide decision-making throughout the project development process, which results in maximizing system performance while minimizing project cost. The next sections will outline the approach designers will take to implement PBPD on TxDOT projects.

2.2 Practical Design Process

2.2.1 What is Practical Design?

Practical Design (PD) is a flexible approach to engineering where projects are evaluated throughout the development lifecycle to look for opportunities to lower project costs. PD looks to address the problem of "how do we do more with less" to maximize system-wide improvements. The approach focuses on maximizing improvements to the roadway system as a whole rather than maximizing improvements to a few locations. PD solutions aren't just limited to reducing construction costs, they also can reduce operating and maintenance costs that are sometimes significantly higher than the original construction cost. PD encourages innovation and flexibility and requires more information and a higher level of analysis when defining and deciding on the most appropriate solution to a particular problem.

Practical Design is not a new or unique concept. Development of 3R design standards was one of the first examples of PD.

PD places emphasis upon solutions that contribute to the overall condition and function of the entire corridor instead of attempting to achieve the most desirable condition for a single project. PD has increased the ability of many states to deliver more cost-effective design solutions, without compromising safety or quality while enhancing the overall transportation system.

2.2.2 Practical Design Principles

The foundation of PD encompasses four principles that should be applied during the planning of all projects. Designers should define and document the **project purpose and need** by specifying the problem to be solved and the future goals of the corridor/system to be achieved by the proposed project. Instead of "designing to the manual", a "designing to the purpose and need" philosophy should be implemented. As a result, **Design Exceptions and Design Waivers are strategically used, but must be adequately supported.**

The typical approach of each project's design has been to improve the specific project as much as possible with the budgeted resources (i.e., using desirable design criteria values). Maximizing improvements in one location may not be the best allocation of resources. PD is an alternative approach to generate the best return of investment. The general principles that guide PD are outlined in the following subsections.

2.2.2.1 Sound Engineering Judgment

Designers should rely upon the application of sound engineering judgment when considering design alternatives and solutions. Design decisions should be driven by common sense, context awareness and sensitivity, and innovation. Over-reliance upon traditional design standards and approaches tends to pre-determine the outcome without adequate consideration for alternative, more practical solutions.

2.2.2.2 "Design Up" Philosophy

Designers should employ a "design up" philosophy to project design. Rather than starting with the "desirable" condition and often being forced to remove items to meet project scope and budget, the designer should consider the existing condition of the facility as the baseline condition, and "design up" from that point to meet the project's purpose and need. Often, the result is a facility that is safe, practical, and less costly than anticipated. Any work beyond the purpose and need of the project is an inefficient use of resources that would likely yield a higher return if invested elsewhere. The designer should perform a cost-benefit analysis of proposed work and its effectiveness to meet the project's purpose and need.

2.2.2.3 Get the Project Scope Right

The purpose and need statement, documented in the DSR serves as the basis for system-wide improvements and individual project development. Any features that do not directly support the purpose and need should be re-evaluated, redesigned, or eliminated. PD reduces the tendency for "scope creep" by designing to, without exceeding, the purpose and need for the project.

2.2.2.4 Safer System Focus

All projects must be as safe, or safer, than the existing condition. Careful consideration should be given to alternative strategies and creative approaches to mitigate safety concerns.

Designers are encouraged to design toward solutions that optimize the condition and performance of the corridor, rather than attempting to achieve perceived individual project perfection.

Targeted investments in specific locations throughout a corridor can have a compounding positive impact upon the overall corridor, more so than a large investment in a single location within the corridor. For example, adding a series of right turn lanes along a corridor versus a full rehabilitation to reduce operational challenges.

Practical Design's purpose is to "build good projects everywhere – rather than perfect projects somewhere."

- Missouri DOT 2005 Strategic Objective to Implement PD

There is not a methodology or flowcharted process for the application of PD to determine what scope and cost of a project will yield an optimized system. However, when combined with the Performance Based Design process explained in the following section, designers will, over time and with experience, develop a feel for the proper application of PD for various project situations.

The PD approach is not optional or elective. Its application is necessary on every project to have a functional transportation for the long term. PD goes beyond saving money on an individual project; it aspires to build the best-performing transportation system with the money available.

2.3 Performance-Based Design Process

2.3.1 What is Performance-Based Design?

Performance-Based Design (PBD) is an evaluation of the performance impacts of highway design decisions. It provides an approach that looks at the outcomes of design decisions as the primary measure of design effectiveness. The **National Cooperative Highway Research Program (NCHRP) Reports 785 Performance-Based Analysis of Geometric Design of Highways and Streets** and **839 A Performance-Based Highway Geometric Design Process** are publications that seek to guide project managers and designers through PBD processes.

- **Report 785** establishes an approach designers can use to evaluate the performance trade-off of different project development and design decisions.
- **Report 839** presents a revised geometric design process providing guidelines based on the project type and the problem or need being addressed.

These reports are recommended reading for all designers and project managers.

2.3.2 Performance-Based Design Guiding Principles

The principles that guide PBD allow designers to create usable, practical, and long-lasting highways and streets.

Intended Outcomes:

The foundational step in PBD is to determine and document a project's purpose, need and problems. This is followed by establishing the desired project outcomes and goals.

Connection to Project Development Process:

Designers should consider the opportunities to apply performance-based analysis during each stage of project development (project initiation, planning, preliminary engineering, and final design). The ability to recognize the benefits of performance-based analysis is greater in the earlier stages of project development (planning and preliminary engineering), and harder to influence project outcomes during final design activities.

Performance Categories:

The goal of PBD is to focus on actual project performance instead of the traditional dimension-based design approach. Areas of performance characteristics include, but are not limited to:

- Quality of service;
- Safety;
- Reliability;
- Accessibility;
- Infrastructure integrity;
- Ease of use;
- Ease of maintenance;
- Visual quality; and
- Fit to context and community.

2.3.3 Performance-Based Design Process

Figure 2-1 illustrates the steps in the general performance-based design process. Chapter 5 of **NCHRP Report 785** gives a detailed process framework for application of performance-based analysis. The application of the framework is organized into three broad project development phases – project initiation, planning, and preliminary engineering.

Project Initiation:

Project Context – Determine the context (rural, rural town, suburban, urban, or urban core) of the project to help identify the facility users and special needs influencing geometric design decisions. See **Chapter 3** for more discussion on context classification and its application.

Intended Outcomes – The results of determining the intended outcomes are the identification of the following:

- Primary and supplementary target users of the project;
- Project objectives and goals; and
- Performance measures to evaluate progress toward the intended outcomes.

Planning:

Geometric Influences – This step identifies the geometric characteristics or decisions that influence a project's performance as it relates to the categories of accessibility, mobility, quality of service, reliability, and safety.

Potential Solutions – Potential solutions should be developed with a particular awareness of what has been learned in previous steps. The solutions can be general concepts early in project development or more detailed, project specific solutions to address a specific need, issue, or challenge.

Preliminary Engineering:

Estimated Performance – Once the appropriate evaluation tool and resources are selected for the given project and performance measures, designers can apply the tool or resource to assess the project alternatives' relative performance.

Financial Feasibility – Financial feasibility approaches include:

- Total construction and maintenance cost of the alternative;
- Cost effectiveness of the alternative; and
- Benefit/cost ratio of the alternative.

Selection – Based on the results of the estimated performance and financial feasibility step, the designer will select a preferred alternative or refine alternatives and re-evaluate their performance.

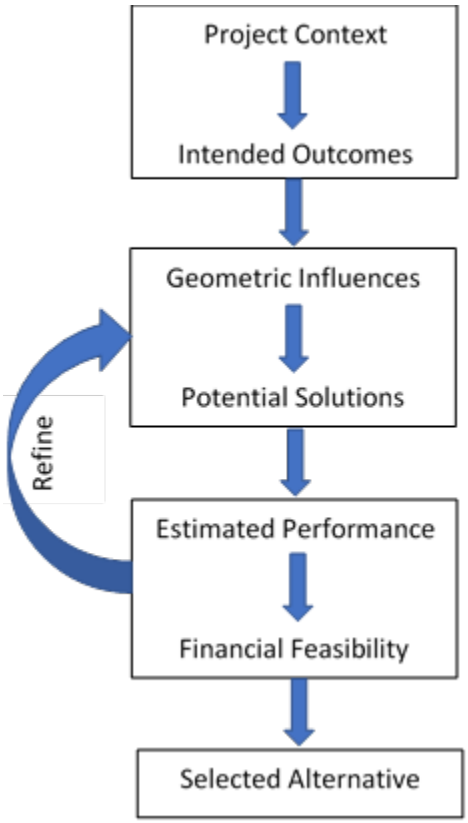


Figure 2-1 Performance-Based Design Process

2.3.4 Resources for Performance Based Analysis

AASHTO's **Highway Safety Manual (HSM)** is the most prevalent resource for performance-based methodology. Other PBD resources have been developed and continue to be developed through research. Such resources include:

- **Interactive Highway Safety Design Model (IHSDM)** – Federal Highway Administration (FHWA);
- **Highway Capacity Manual (HCM) – Transportation Research Board (TRB)**, 2016;
- **NCHRP Report 687 Guidelines for Ramp and Interchange Spacing,** 2011;
- **NCHRP Report 839 A Performance-Based Highway Geometric Design Process**, 2016; and
- **Traffic and Safety Analysis Procedures Manual (TSAP)** – Texas Department of Transportation (TxDOT), 2024.

2.4 Performance Based Practical Design

2.4.1 What is Performance Based Practical Design?

What is PBPD?	
What PBPD is:	**What PBPD is not:**
Grounded in performance management	New regulation or requirement by FHWA
Exercises engineering judgement to address a project's purpose and need	New version of Value Engineering (VE)
Uses appropriate performance-analysis tools	Replacement for Context Sensitive Solutions (CSS)
Considers both short- and long-term project and system goals	Opportunity to disregard long-term needs
-	Compromise on safety or user needs to save money

Performance Based Practical Design (PBPD) is the process of performance evaluations, as previously discussed in **Section 2.3**, during project development to guide alternative selections and, ultimately, geometric design decisions. PBPD modifies the traditional dimensional design approach to a "design up" approach – using engineering judgement and practical design to build up the improvements from the existing conditions to meet both project and system objectives.

PBPD is an approach to decision-making that encourages engineered solutions rather than reliance on maximum values or limits found in design manuals. PBPD is making practical decisions but making sure those decisions are based on an objective analysis of the relevant data.

FHWA discusses notable attributes of PBPD to be:

- PBPD focuses on performance improvements that benefit both project and system needs.
- Agencies make sound decisions based upon performance analysis.
- By scrutinizing each element of a project's scope relative to value, need and urgency, a PBPD approach seeks a greater return on infrastructure investments.

PBPD Used In All Phases Of The Project Lifecycle	
Planning	**Preliminary Engineering & Design**
Define system and project purpose and need	Start with existing conditions as the baseline
Develop concise project objectives from the purpose and need	Engineer solutions that meet the purpose and need – not just meet standards
Consider project in the context of the system	Evaluate tradeoffs based on objective analysis of data
Address all users	Support with design exceptions if needed

- PBPD strengthens the emphasis on planning-level corridor or system performance needs and objectives when planning, scoping, and developing individual projects.
- PBPD can be implemented within the Federal-aid Highway Program regulatory environment utilizing existing flexibility.

- PBPD does not eliminate, modify, or compromise existing design standards or regulatory requirements.
- PBPD should consider multiple design and operating solutions to find the combinations that best addresses the project objectives.

Additional guidance on PBPD and Benefit/Costs Analyses can be found in the **TSAP**. Examples of PBPD can also be found in the following references:

- FHWA PBPD case studies;
- Demonstrating Performance-Based Practical Design through Regional Performance-Based Planning – Federal Highway Administration (FHWA);
- Demonstrating Performance-Based Practical Design through Alternative Intersections – Federal Highway Administration (FHWA);
- Demonstrating Performance-Based Practical Design through Analysis of High-Occupancy Toll Lanes – Federal Highway Administration (FHWA); and
- FHWA-HIF-16-030 - Federal Highway Administration (FHWA).

2.4.2 Performance Based Practical Design Application

A PBPD approach should be utilized in all projects commensurate with the scope of the project. For 4R projects, the PBPD process can be used to optimize the design elements of a project that will provide the most safety at a better cost. The respective benefit-cost evaluation may use the tools and methods as described in the **TSAP**, or other available approved tools. The level of analysis should be commensurate with the size and scope of the project. These tools will use a present-day cost and value of benefits to develop a Benefit/Cost ratio (including safety benefits). Generally, B/C ratios of 2.0 or higher represent a significant benefit, while ratios between 2.0 and 1.0, though considered positive, require additional sensitivity analyses to determine whether the identified upgrades are sufficiently beneficial for the project and TxDOT. B/C values below 1.0 should generally not be considered. For those elements that it is determined from a Benefit-Costs standpoint that the design minimums for a particular element will not be achieved. The PBPD analysis would serve as the basis for any needed design exceptions or design waivers.

Design guidelines for 3R projects have been developed to allow for greater design flexibility. 3R design decisions should be based on an assessment of the safety and traffic operational performance of the existing road and the cost-effectiveness of potential design improvements. **Chapter 10** covers the application of PBPD for 3R projects and uses as the primary basis **NCHRP Report 876 Guidelines for Integrating Safety and Cost-Effectiveness into Resurfacing, and Rehabilitation Projects (3R)**, and **NCHRP web-only document 244 (companion to NCHRP 876)** which has spreadsheet tools for comparing several alternatives or combinations of alternatives. There are additional tools and methodologies as referenced in the **TSAP**. As with 4R projects, the Benefit-Costs analyses

would serve as the basis for any needed design exceptions or design waivers.

Section 10.2 also covers roadway tradeoff cross-section optimization guidance for 4 lane and 2 lane roadways (a form of PBPD) that may be applicable to certain 4R projects as well.

The TxDOT Safer by Design Tools (found on the **Safer by Design Tool webpage** on TxDOT.gov), required for all applicable construction projects, is an additional tool available which utilizes proven safety measures based on the predictive method included in the **HSM**. This tool is unique to Texas roadway facilities and provides quantitative scores specific to how a change in a roadway design may directly influence the safety performance of a roadway.

2.4.3 What is the Difference between PBPD and Value Engineering?

FHWA views PBPD as the next step in the evolution of project development and design decision-making. Practical Design, Context Sensitive Solutions, Value Engineering and other similar initiatives and strategies are considered key components of a successful PBPD program.

Value Engineering:

- Method to determine the most cost-effective way to achieve proposed improvements;
- Typically focuses on maximizing project improvements; and
- Performed by multidisciplinary team not involved in the project.

PBPD:

- Method to determine the most cost-effective way to achieve the project objectives;
- Focuses on maximizing roadway system improvements, system safety, and TxDOT's strategic goals; and
- Performed by project team and project stakeholders.

Chapter 3
Context and Facility Type Considerations

3.1 Overview

TxDOT's functional and context classification system describes the general characteristics of the land use, development patterns, and roadway connectivity along a highway. These elements serve as indicators for the type of users that will likely utilize the roadway so that designers can optimize the roadway design. The system provides additional contexts beyond urban, rural, and suburban and facilitates modes of travel other than the automobile (e.g. bicycles, pedestrians, transit, and freight).

Functional classification is a federally-mandated categorization of roadways based on the level of traffic service and degree of access they provide. Updates to functional classifications are evaluated collaboratively by the TPP Division, TxDOT Districts, MPOs, and FHWA staff and can occur on an as-needed basis as roadways are constructed and/or expanded.

Context classification provides the roadway designer with an understanding of the function of the roadway within its current and expected future context and the needs of the potential roadway users. The use of an expanded functional classification system (FCS) that incorporates additional contextual definitions provides a more dynamic range of design elements and guidance needed to balance competing project needs in addition to mobility and access.

The use of context classification in addition to functional classification, to determine criteria for roadway design elements is consistent with national best practices and direction, including the **AASHTO's A Policy on Geometric Design of Highways and Streets** and **NCHRP Report 855**. This guidance has been developed to provide a flexible framework, in addition to the existing functional classification of roadways, to facilitate optimal geometric design solutions that take into account the environmental context, road functions, and user needs. The principal objective of the expanded functional and context classification system is providing enhanced information to designers to better inform the design decision process beyond previous methods.

Proper contextual roadway designs require an understanding of the function of the roadway within its current and expected future context and the needs of the potential roadway users. The guidance included in this chapter provides an explanation and proposed methodology for use of the expanded functional classification system.

One of the first steps in project scoping is to prepare a "Purpose and Need Statement" for the project and to determine where the project falls in the design framework shown in **Figure 3-1**.

Nineteen of these 20 roadway types are commonly found on the TxDOT System of Highways. The exception is freeways in a rural town context which is unlikely to occur.

This guidance incorporates the framework for geometric design based primarily on:

- A Functional Classification System that characterizes roadways by their position in the transportation network and the role provided to various user types; and
- A context classification System that characterizes roadways by their surrounding environment and how the roadway fits into the community.

Functional Class	Context Class				
	Rural	Rural Town	Suburban	Urban	Urban Core
Freeway					
Arterial					
Collector					
Local					

Note: The shaded cell, representing a freeway in a rural town context, is unlikely to occur.

Figure 3-1: Context Classification Categories

3.2 Functional Classification System

One of the initial steps 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. The functional classification of roadways can be viewed on the **Statewide Planning Map** that is maintained by TxDOT's Transportation Planning and Programming Division (TPP) and is also shown in TxDOTConnect. 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. Key characteristics of each roadway type are defined in **Table 3-1**. **Figure 3-2** illustrates the hierarchy of roadways within the functional classification system.

Table 3-1: Roadway Functional Classification System

Functional Classification	Network Importance	Characteristics
Interstates / Freeways Expressways	Corridors of national importance providing long-distance travel	• Limited Access • Through traffic movements • Long-distance traffic movements • distance traffic movements • Primary freight routes • Possible transit network support • No pedestrian or bicycle traffic • Guided primarily by FHWA design standards
Principal Arterial	Corridors of regional importance connecting large activity centers[1]	• Non-access controlled • Through traffic movements • Long distance traffic movements • Primary freight routes • Some segments may include pedestrian and bicycle movements.
Minor Arterial	Corridors of local importance connecting activity centers[1]	• Connections between local areas and network principal arterial roadways • Connections for through traffic between arterial roadways • Access to public transit and through movements • Pedestrian and bicycle movements
Major Collector	Roadways providing connections between arterials to local roads and streets	• Traffic with trips ending in a specific area • Access to commercial and residential centers • Access to public transportation • Pedestrian and bicycle movements
Minor Collector	Roadways providing connections between major collectors to local roads and streets	• Fewer lanes, lower ADTs and more access points than major collectors • Access to public transportation • Pedestrian and bicycle movements
Local	All other roads	• Direct property access – residential and commercial • Pedestrian and bicycle movements

Notes:

1. An activity center is a concentrated area with a high density of diverse land uses and activities that generate significant traffic. These areas often include a mixture of commercial, recreational, institutional, entertainment, and tourism facilities. Examples include transit stations, shopping malls, park-and-ride facilities, business districts, and University campuses.

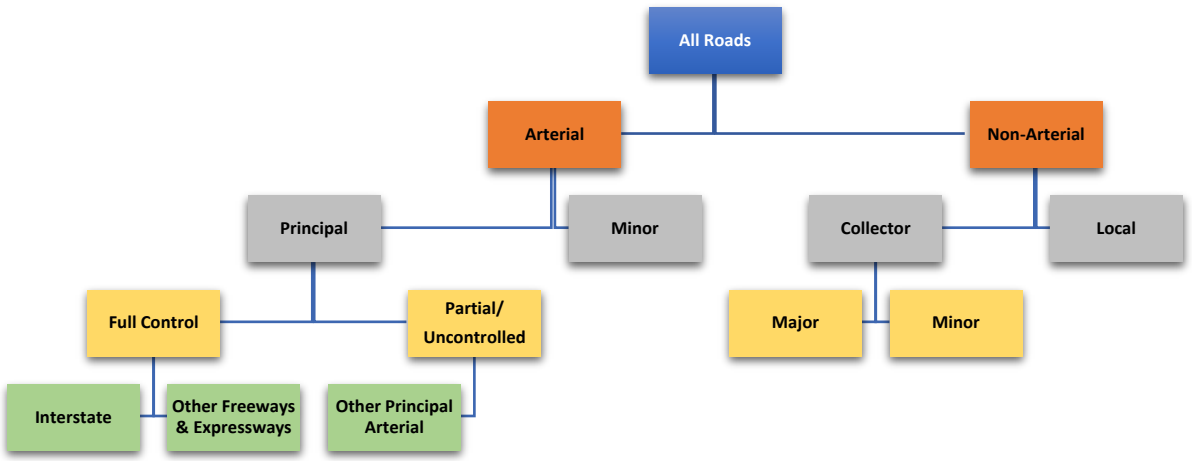


Figure 3-2: Functional Classification System and Hierarchy

Source: FHWA Highway Functional Classification Concepts, Criteria and Procedures 2023 Edition

3.3 Context Classification System

The context classification system broadly identifies various environments in Texas as illustrated in **Figure 3-3**. This figure should not be taken as a literal depiction implying all roadways will have every context classification or that context classifications occur in the sequence shown. However, it is good practice that a freeway connects to an arterial, an arterial connects to a collector and a collector connects to a local street.

Identifying the context classification of a roadway is a required step in the planning phase of the project development process. It is essential to identify the appropriate design criteria in relation to the functional class and the intended use of the roadway. The project development process is further discussed in the **PDP**.

Context classification is a supplemental means to identify the anticipated users of the roadway and the needs of those users. It is important to understand the user type and needs early in the project life cycle. The context classification for a roadway is determined during project scoping and is documented in the DSR.

Understanding the needs of all users in the early phases of a project assures that the project scope of services defines all necessary improvements. The project context classification should be determined and/or re-evaluated at the beginning of each phase of project development including planning, preliminary engineering, and final design.

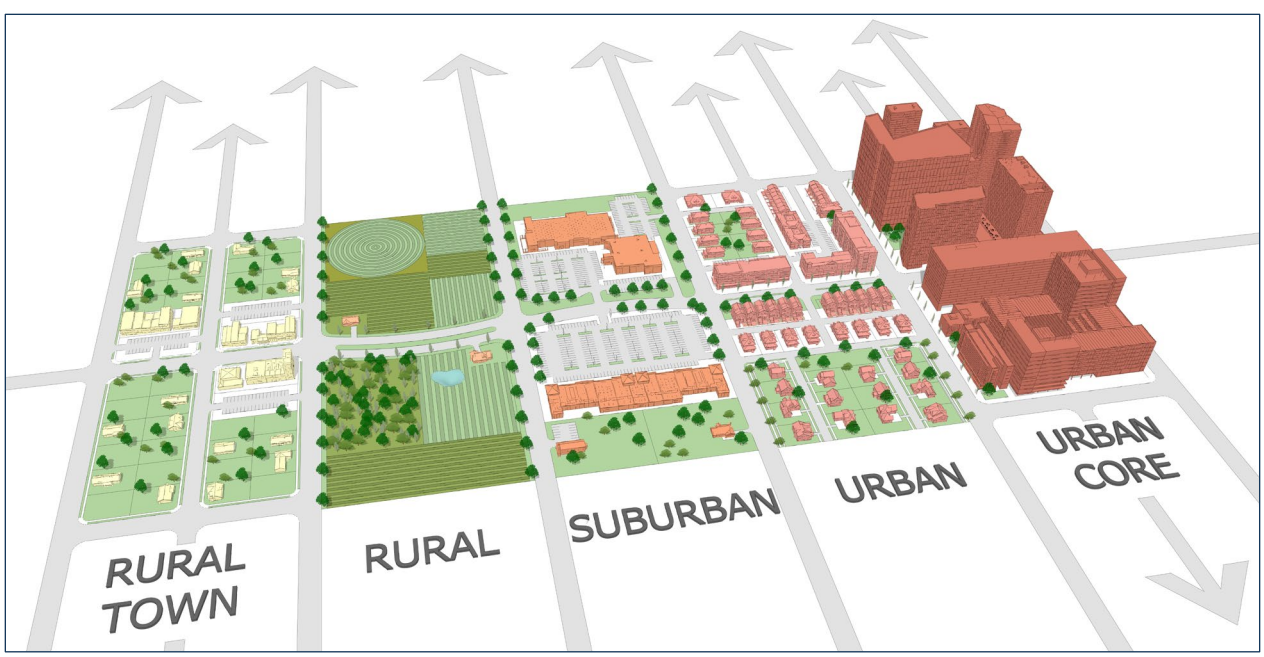


Figure 3-3: TxDOT Context Classifications

3.3.1 Determining Context: Factors and Process

The general framework for the demarcation between various contexts is shown in **Figure 3-3**. **Table 3-2** describes the primary characteristics of the respective context:

- Development density (existence of structures and structure types);
- Land uses (primarily residential, commercial, industrial, and/or agricultural);
- Building setbacks (distance of structures to adjacent roadways);
- Sidewalk and parking accommodations;
- Transit and freight accommodations; and
- Intersection/driveway density.

The context classification process does not rely solely on quantitative data or analysis (e.g., persons per square mile or building square footage), but can be used where broad comparative development differences are encountered. The context classification and functional class decisions become the starting point for geometric design choices because these choices address the modes to be accommodated and their interactions. The context category should be assigned based upon generally observed conditions within and among the determining factors as well as the consideration of expected future changes in land use patterns. The roadway planning and design process should take into account anticipated future context conditions that are often defined through State, regional, and local planning documents.

Functional class drives the function of the roadway, context classification drives the constraints of the roadway.

An Urban area, as defined by FHWA, has a population of 5,000 or greater (based on the latest Census data); all areas below a population of 5,000 are currently classified as rural. These areas can be seen on the **Statewide Planning Map** that is maintained by TPP . Per FHWA guidance, after each decennial census, TPP, in partnership with the MPOs and Districts, expand the urban areas for transportation planning purposes. This is generally completed about 3 years after the latest decennial census

In a recent effort, TPP (in collaboration with TTI) has developed layers in the **ArcGIS platform** that will visually show context in a baseline year, 10-year forecast, and 20 year horizon down to the Traffic Analysis Zone (TAZ) level. These layers are based on the travel demand models trip generation forecasts for the respective MPO area. These layers will provide designers with an additional tool to help distinguish between urban core, urban, suburban, and rural contexts; the layers also have a finer degree of gradation for areas that are on the cusp between Urban Core and Urban, as well as between Suburban and Rural. This tool is located on the **TxDOT ArcGIS platform** under the layer (TPP_TDM_AreaType). Contact TP&P (GIS Management Section) for access credentials.

Note that a rural town does not have a specific quantitative definition and thus may reside in either an urbanized or rural area.

The subsequent sections in this chapter further describe and depict urban core, urban, suburban, rural town, and rural contexts. Note that urban core, urban, suburban, and rural town are considered "urbanized" contexts.

Table 3-2: Context Classification Categories and the Primary Factors used to Determine the Context of Roadways.

Context Classification	Development Density	Land Use	Setbacks	Sidewalk and Parking	Transit and Freight	Intersection / Driveway Density
Rural	Lowest (few houses or other structures)	Agricultural, natural resource preservation, and outdoor recreation uses with some isolated residential and commercial	Usually large setbacks	No sidewalks or on-street parking	Little transit; Moderate freight	1-10/mile
Rural Town	Low to medium (single-family houses and other single purpose structures)	Primarily commercial uses along a main street with some adjacent single-family residential	Predominately small setbacks	Some on-street parking and some sidewalks	Little transit; moderate freight	>30/mile
Suburban	Low to medium (single- and multifamily structures and multistory commercial)	Mixed residential neighborhood and commercial clusters (includes town centers, commercial corridors, big box commercial and light industrial)	Predominately large setbacks	Some sidewalks and no street parking	Little transit; moderate freight	10-30/mile
Urban	High (multistory, low-rise structures with designated off-street parking)	Mixed residential and commercial uses, with some institutional, industrial, and prominent destinations	Mixed setbacks	On-street parking and sidewalks	High transit; moderate freight	>30/mile
Urban Core	Highest (multistory and high-rise structures)	Mixed commercial, residential, and institutional uses within and among predominately high-rise structures	Small setbacks	On-street parking and sidewalks and pedestrian plazas	High transit; low freight	>30/mile

3.3.2 Rural Context

The level of development in rural contexts ranges from minimal (no development) to light (a few structures). Most land is used for outdoor recreation, agriculture, farms, and resource extraction. Unincorporated areas may include a few residential and commercial structures. Building setbacks are typically large but can taper near intersections and crossroads where development is present. Population densities are low. Speed expectations for drivers are higher, with infrequent driveways or intersecting roads and few slowing or turning vehicles. Examples of typical rural areas are shown below.

Typical characteristics are:

- Few houses or structures;
- Widely dispersed residential, commercial, or industrial land uses;
- Long-distance bicyclists using highway shoulders:
- Large building setbacks; and
- Undeveloped land, farms, large outdoor recreation areas, or low densities of other types of development.



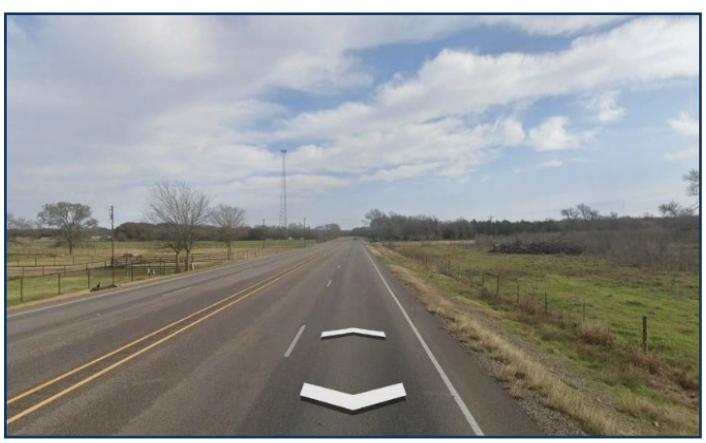


Figure 3-4: Example Roadways in Rural Context
Source: Google Maps

3.3.3 Rural Town Context

A rural town does not have a specific population definition and thus may reside in either an urbanized or rural area as currently defined in the **Statewide Planning Map**. Rural town contexts feature small concentrations of developed areas, including historic towns, surrounded by rural areas, undeveloped areas, and natural areas. Building densities are lower, but often resemble urban development patterns and consist of low-rise structures (one or two stories). Most land use is residential or commercial. Relatively compact residential uses are expected, and schools may be present. Building setbacks are small and much narrower than surrounding areas. A basic street

network is usually in place. Rural highways change character where they enter a rural town. Driver speed expectations are lower, with a higher likelihood of encountering slowed or turning vehicles, pedestrians, and bicyclists. Examples of typical rural town areas are shown below.

Typical characteristics are:

- Low development densities with mixed land uses;
- Some on-street parking;
- Average building setbacks less than 50 ft;
- Average driveway densities greater than 30 driveways/mile on both sides of the road;
- May include residential neighborhoods, schools, industrial facilities, and commercial main street business districts; and
- Some pedestrian and bicyclist activity, often with sidewalks and marked crosswalks in some locations.



Figure 3-5: Example Roadway in Rural Town Context
Source: Google Maps

3.3.4 Suburban Context

Suburban contexts include an array of commercial and residential uses, low to medium densities, medium to large building setbacks (to accommodate off-street parking), and large block sizes. Suburban areas are usually connected to urban areas. Buildings tend to be single story or low-rise structures, with off-street parking in commercial areas. Residential areas are populated by mostly single-family (with some multifamily) housing and some neighborhood parks. Drivers have higher speed expectations than the urban contexts, but lower speed expectations than the rural contexts. Suburban areas are generally short sections, approximately 1 – 3 miles in length, and transition between rural and urban contexts. The suburban context may be more applicable in areas of anticipated future growth that eventually will become an urban context based on future land-use expectations within the expected design life of the facility. Examples of typical suburban areas are shown below.

Typical characteristics are:

- Low-to medium density development;
- Mixed land uses with single-family residences, some multi-family residential structures, and/or nonresidential development including mixed town centers, commercial corridors, big box commercial stores, light industrial development;
- Building setbacks are varied;
- Driveway densities between 10 - 30 driveways/mile on both sides of the road;
- Mostly off-street parking; and
- Pedestrians and bicyclist activity; may or may not have sidewalks and marked crosswalks.

The term “town centers” refers to areas within a town or city where commercial, social, and civic activities are concentrated. Town centers typically feature shops, restaurants, offices, and public spaces.



Figure 3-6: Example Roadway in Suburban Context
Source: Graham-Smith Partners – AASHTO Green Book 7th Edition

3.3.5 Urban Context

Urban contexts are medium to high density and consist mainly of multistory and low-to medium-rise structures for residential, commercial, and institutional use. Many structures are mixed use: commercial, residential, and parking. Building setbacks are typically small with pedestrian-focused building orientations. Urban contexts generally experience lower driver speed expectations, with a higher likelihood of encountering stopped or turning vehicles, pedestrians, and bicyclists. Examples of typical urban areas are shown below.

Typical characteristics are:

- High-density development;
- Some on-street parking;
- Varied building setbacks Multi-story and low-to medium-rise structures for residential, commercial, parking and educational uses;
- Light to heavy industrial land use;
- Prominent destinations with specialized structures, (e.g., large theaters, sports facilities, or conference centers);
- High levels of pedestrian and bicyclist activity, with nearly continuous sidewalks and marked crosswalks;
- Higher density of transit stops and routes;
- Driveway densities greater than 30 driveways/mile on both sides of the road;
- Minor commercial driveway densities of 10 driveways/mile or greater; and
- Major commercial driveways.

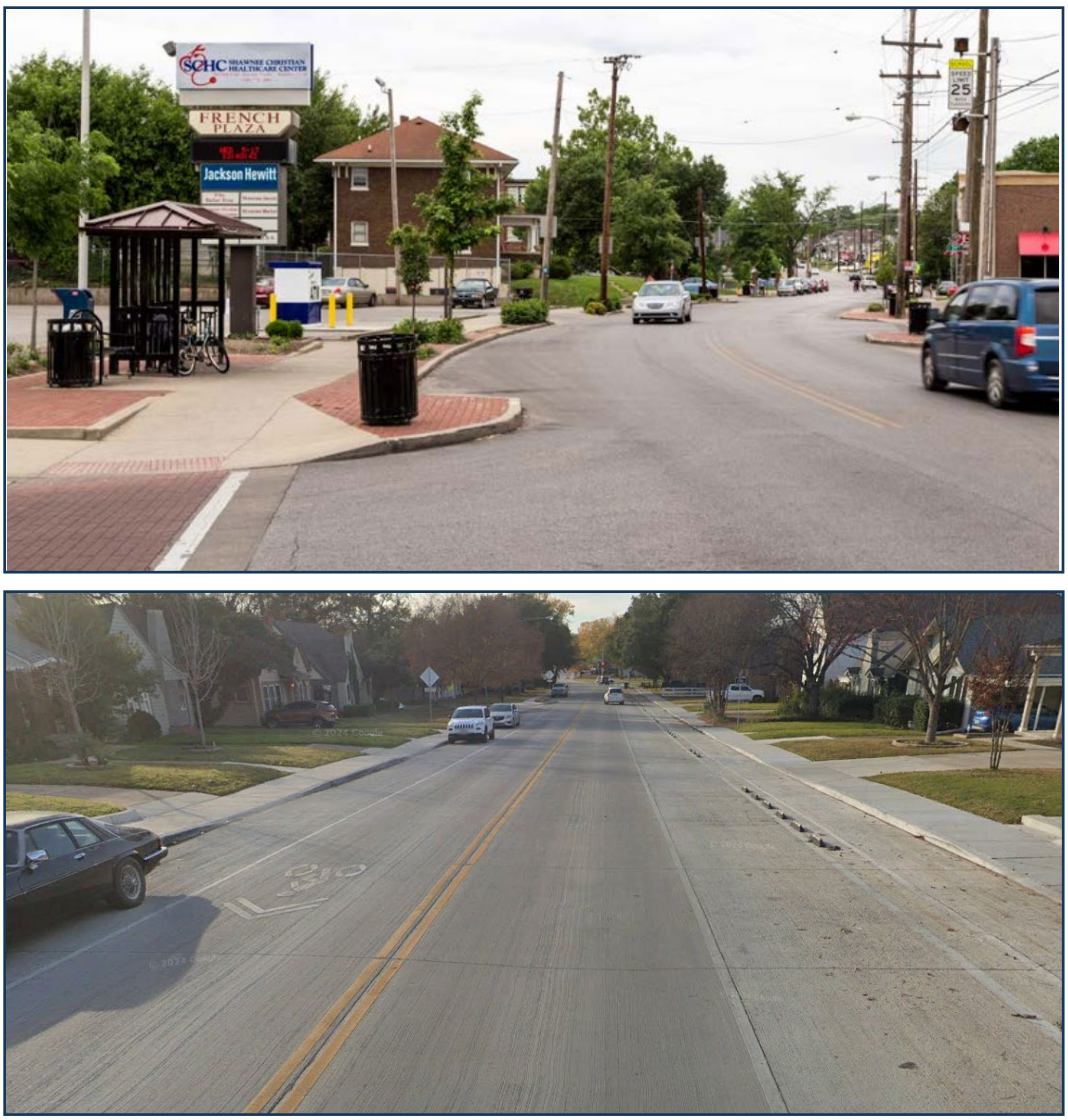


Figure 3-7: Example Roadway in Urban Context

Source: Graham-Smith Partners – AASHTO Green Book 7th Edition (Top Image)

3.3.6 Urban Core Context

Urban core context houses the highest level of development density with its mixed residential and commercial uses accommodated in high-rise structures. While there may be some on-street parking, it is usually very limited and time restricted. The area is accessible to automobiles, commercial delivery vehicles, and public transit. Bicycle facilities and transit corridors are common. Structures may have multiple uses, and setbacks are not as generous as in the surrounding urban area. Examples of typical urban core areas are shown below.

Typical characteristics are:

- High-density development;
- On-street, time-restricted parking, or off-street in parking structures;
- Small building setbacks;
- Driveway densities greater than 30 driveways/mile on both sides of the road;
- High pedestrian, bicycle, and transit use; and
- Multi-story and high-rise structures for residential, commercial, and educational uses.

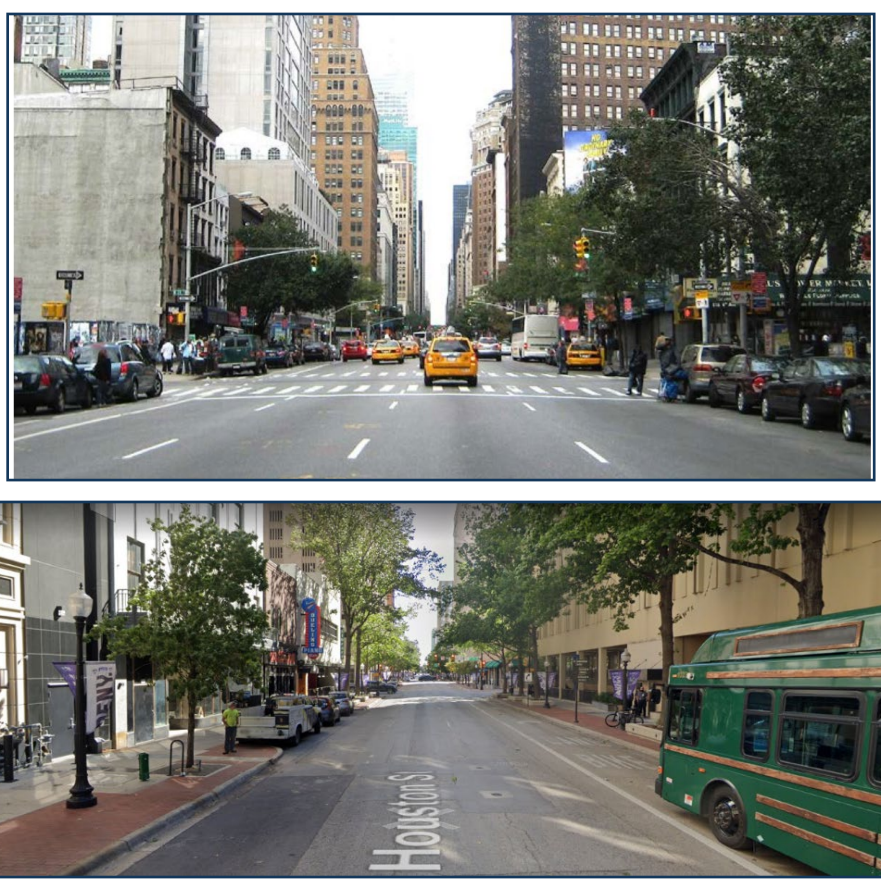


Figure 3-8: Example Roadway in Urban Core Context

Source: Graham-Smith Partners – AASHTO Green Book 7th Edition (Top Image); Google Maps (Bottom Image)

3.4 Roadway Users/Transportation Networks

While functional and context classification provides a general understanding of the type and activity level of different users, additional data related to travel patterns and user demographics assists to identify the needs of users and inform solutions to be considered to meet users' needs. The anticipated users of a roadway and travel patterns of different transportation networks should be determined early in the planning phase of the project development process.

The context classification process includes anticipated future context conditions that are often defined in State, regional and local planning documents. The assessment of long-range transportation plans, land use studies and local street design guidelines as well as engagement with stakeholders provides a better opportunity to determine the likelihood of future context change.

Additionally, understanding the roadway network and the needs of all users is essential in the selection of the roadway context. The use of context classification incorporates all transportation networks including automobile, bicycle, and pedestrian networks allowing for a more complete approach in designing roadways. The application of these networks to context classification determination is essential to define the level of accommodation needed for the different modal users.



Automobile Network - The basic roadway functions are defined on their network function and connectivity as discussed in **Section 3.2**. Various user needs are identified during project planning to balance and make necessary tradeoffs among design elements to develop contextually appropriate multimodal solutions.



Bicycle Network - The primary consideration of bicycle facilities for context classification is the level of separation between motorized and bicycle traffic. See **Chapter 18** for application of bicycle considerations and design criteria in different contexts.



Pedestrian Network – The primary consideration of pedestrian facilities for context classification is its width. See **Chapter 19** for application of pedestrian considerations and design criteria in different contexts.



Transit Network - It is important to incorporate transit facilities into the overall roadway network so that they can be considered in the context of the overall transportation network. The designer should perform close coordination with transit agencies to define where transit either exists or is anticipated to be located. See **Chapter 22** for additional discussion on this topic.



Freight Network - Freight networks can be characterized based upon the frequency and size of expected freight traffic. The designer should study the land use to identify industrial centers, multimodal ports, manufacturing, and commercial areas to determine existing and future freight networks. The preferred supply and delivery routes that connect the origin and destination of the freight traffic should be identified to determine the roadway context. See **Chapter 21** for additional discussion.

The safety of all road users must be equitably addressed in the selection of roadway type and use. The road system should be planned, designed, and operated to be forgiving of inevitable human mistakes, so that serious injury outcomes are unlikely to occur.

Table 3-3 provides a menu of data sources that the designer can use to identify different needs for different users. Not all data in this table will be required for all projects. **The data collected for a project should be tailored to the scale of the projects and the users the project needs to serve.**

Table 3-3: Potential Data to Determine User Needs by Mode

Mode	Data	
 Automobile	• Design traffic (existing and future AADT, K-factor, directional distribution, and traffic growth projections • Trip length, origin/destination patterns • Turning movement counts • Posted and operating speeds • Signal timing	• Location and availability of parking • Crash data • Lighting levels • Pavement condition • Existing and future land use, building form and site layout, development scale and pattern. • Evacuation Routes
 Bicycles	• Local and regional bicycle network • Posted and operating speeds • Vehicular traffic volumes • Number of vehicular travel lanes • Location and availability of bicycle parking • Bicycle user type • Existing bicycle facility characteristics (location, width, obstacles, separation) • Bicyclist counts	• Crash data • Location of destinations • Lighting levels • Pavement condition • Existing and future land use, building form and site layout, development scale and pattern • Transit Connections
 Pedestrians	• Location of signalized pedestrian crossings • Location of marked or signed pedestrian crossing • Posted and operating speeds • Vehicular traffic volumes • Existing sidewalk characteristics (location, width, condition, obstacles, gaps, separation from vehicles) • Intersection ramps and alignment/ADA compliance	• Utilities location • Existing landscape buffer and shade trees • Pedestrian counts • Crash data • Lighting levels • Existing and future land use, building form and site layout, development scale and pattern • Existing and future pedestrian generators (e.g., schools, parks, transit stops) • Transit Connections
 Transit	• Existing and future transit routes and stops • Transit service headways • Location and infrastructure at transit stops • Sidewalk and bicycle facility connection to transit stops • ADA compliant transit stops	• Existing and future transit generators and attractors • Existing and projected ridership (route or stop level) • Type of transit technology • Trip lengths, origin/destination patterns
 Freight	• Designated truck routes • Truck volumes • Vehicle classification counts	• Existing and future location of industrial land uses or other generators of freight trips • Freight loading areas/truck parking • Hazmat Routes

Table 3-4 shows the typical user priorities for each possible mode. Typical uses are based on current traffic trends and existing networks, and they should not be viewed as modal accommodation for each context and roadway-type combination.

Table 3-4: Context Classification Matrix and Typical User Priorities

Functional Class	Context Class				
	Rural	Rural Town	Suburban	Urban	Urban Core
Principal Arterial					
Minor Arterial					
Collector					
Local					

Low 

Medium 

High 

3.5 Speed, Mobility and Access Considerations

Issues associated with users should be considered to achieve a proper contextual design solution. Driver accommodation is one area that defines the context-roadway interaction for drivers considering **target speed** and the balance between **mobility** and **access**.

3.5.1 Target Speed

The use of target speed for roadways recognizes the influence of driver desire and expectation. The target speed is the operating speed that the designer intends for the drivers to use. A goal of the expanded functional and context classification system is to develop a facility where the target speed is in the proximity of the design speed. The target speed should consider both existing and future volumes and contexts. **Section 4.2.4** provides additional guidance for the determination of target speed.

3.5.2 Mobility and Access

Mobility is defined as function of congestion level:

- Low: (congested conditions);
- Medium: (some congestion); and
- High: (no congestion; free flow).

Mobility levels are based on general concepts of the level of service (LOS) for a facility and correspond to broad values of all roadways.

Access is defined as the frequency of driveways or intersections and are grouped into three categories based on distance between access points:

- Low: (> 0.75 mile)
- Medium: (0.75 – 0.25 mile)
- High: (< 0.25 mile)

Definitions of mobility and access are derived from **NCHRP Web-Only Document 230: Developing an Expanded Functional Classification System for More Flexibility in Geometric Design**.

Table 3-5 shows how driver expectations change based on the different combinations of context and roadway functional classification. Refer to Chapter 1 of the **Access Management Manual** for additional information on this topic.

Table 3-5: Speed, Mobility, and Access Considerations

Functional Class	Context				
	Rural	Rural Town	Suburban	Urban	Urban Core
Freeway	Speed: High Mobility: High Access: Low	N/A	Speed: High Mobility: High Access: Low	Speed: High Mobility: High Access: Low	Speed: High Mobility: High Access: Low
Arterial	Speed: High Mobility: High Access: Med	Speed: Low/Med Mobility: Med Access: High	Speed: Med Mobility: Med Access: Med	Speed: Low/Med Mobility: Med Access: Med/High	Speed: Low Mobility: Med Access: Med/High
Collector	Speed: High/Med Mobility: Med Access: Med	Speed: Low Mobility: Med Access: High	Speed: Med Mobility: Med Access: High	Speed: Low Mobility: Med Access: High	Speed: Low Mobility: Med Access: High
Local	Speed: Med Mobility: Med Access: Med	Speed: Low Mobility: Med Access: High	Speed: Low Mobility: Low Access: High	Speed: Low Mobility: Low Access: High	Speed: Low Mobility: Low Access: High

3.6 Implementation of Functional and Context Classification System

The following outlines a general approach to implementing functional and context classification for projects:

1. Define Context and Roadway Type:
    - Determine functional classification of roadway (freeway, arterial, collector, local). *Note, if different than existing, coordinate with TPP*
    - Determine context of roadway (rural, rural town, suburban, urban, urban core)
    - If the roadway consists of two or more contexts (i.e., rural and suburban), develop criteria for each segment
2. Identify roadway users and needs:
    - Determine needs for automobiles, trucks, bicyclists, and pedestrians
    - Identify potential user ranges for speed, access and mobility
3. Consider needs for transit and freight accommodations
4. Develop Purpose and Need Statement
5. Develop cross section alternatives for consideration (Use TxDOT Safer by Design tool to assess alternatives)

Chapter 4
Basic Design Criteria

4.1 Overview

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 for motorized transportation users.

This chapter discusses the basic design criteria that is applicable to all 4R projects and includes the following sections and subsections:

4.1 Overview

4.2 Speed

- 4.2.1 Operating Speed;
- 4.2.2 Running Speed;
- 4.2.3 Design Speed;
- 4.2.4 Target Speed; and
- 4.2.5 Posted Speed.

4.3 Traffic Characteristics

- 4.3.1 Traffic Volume;
- 4.3.2 Design Vehicles;
- 4.3.3 Control Vehicles; and
- 4.3.4 Highway Capacity.

4.4 Access Management

4.5 Safety

4.6 Terrain

- 4.6.1 Level Terrain;
- 4.6.2 Rolling Terrain; and
- 4.6.3 Mountainous Terrain.

4.7 Horizontal Alignment

- 4.7.1 General Considerations;
- 4.7.2 Curve Radius;
- 4.7.3 Superelevation Rate;
- 4.7.4 Superelevation Rates on Low-Speed Rural Town, Suburban, Urban and Urban Core Facilities;
- 4.7.5 Superelevation Rate on Rural and High-Speed Suburban, Urban, and Urban Core Facilities;
- 4.7.6 Superelevation Transition Length;
- 4.7.7 Superelevation Transition Placement;
- 4.7.8 Superelevation Transition Type; and
- 4.7.9 Sight Distance on Horizontal Curves.

4.8 Vertical Alignment

- 4.8.1 Grades;
- 4.8.2 Vertical Curves;
- 4.8.3 Grade Change without Vertical Curves;
- 4.8.4 Vertical Alignment at Railroad Crossings;
- 4.8.5 Vertical Alignment at Intersections; and
- 4.8.6 Vertical Clearance.

4.9 Combination of Horizontal and Vertical Alignment

4.10 Cross Sectional Elements

- 4.10.1 Through Travel Lanes;
- 4.10.2 Speed Change Lanes;
- 4.10.3 Shoulders;
- 4.10.4 Pavement Cross Slope;
- 4.10.5 Pavement Taper Lengths;
- 4.10.6 Rumble Strips;
- 4.10.7 Clear Zone;
- 4.10.8 Lateral Offset;
- 4.10.9 Border and Outer Separation;
- 4.10.10 Right of Way Width;
- 4.10.11 Roadside Design;
- 4.10.12 Curbs and Curb/Gutter;
- 4.10.13 Side Slopes;
- 4.10.14 Safety Edge;
- 4.10.15 Medians;
- 4.10.16 Parking;
- 4.10.17 Retaining Structures; and
- 4.10.18 Utilities and Easements.

4.11 Sight Distance

- 4.11.1 Stopping Sight Distance;
- 4.11.2 Decision Sight Distance;
- 4.11.3 Passing Sight Distance;
- 4.11.4 Intersection Sight Distance; and
- 4.11.5 Sight Distance at Under-Crossings.

4.12 Drainage Facilities

- 4.12.1 Design Treatment of Cross Drainage Culvert Ends;
- 4.12.2 Small Culverts;
- 4.12.3 Intermediate Culverts;
- 4.12.4 Large Culverts;
- 4.12.5 Bridge Class Drainage Culverts;
- 4.12.6 Parallel Drainage Culverts; and
- 4.12.7 Side Ditches.

4.13 Environment

4.14 Human Factors

- 4.14.1 Older Drivers & Pedestrians

4.15 Roadway Intersecting Department Projects

4.2 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.

4.2.1 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.

4.2.2 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 (LOS) 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 the **HCM**.

4.2.3 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.**

Design speed is different from other controlling criteria in that it is a design control, rather than a specific design element. The design speed establishes the range of design values for many of the other geometric elements of the highway.

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.

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 topography and other factors are such that drivers are likely to travel at high speeds.

Some 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.

4.2.4 Target Speed

The working definition for “target speed” is the operating speed that the designer intends for drivers to use. In mixed-use urban areas, the **target speed** is the highest speed at which vehicles should operate on a low-speed thoroughfare in a specific context, consistent with the level of multimodal activity generated by adjacent land uses, to provide both mobility

for motor vehicles and a desirable environment for pedestrians, bicyclists, and public transit users. In general, the target speed for rural roadways should be on the higher end of the design speed range.

The target speed approaching roundabouts, or some other alternative types of intersections, should be properly stepped down to accommodate the desired lower target speed within the intersection. See **Chapter 14** Alternative Intersections for additional information on when target speeds should be stepped down as drivers approach these types of intersections.

The findings of **NCHRP 15-76 – Designing for Target Speed**, which is to be completed in late 2024, will be used as an additional resource to provide recommendations in the roadway design process.

In general, the Target Speed for rural roadways should be on the higher end of the Design speed range. In general, for urban roadways, consider starting with Target speeds on the lower end of the Design speed range.

4.2.5 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.

4.3 Traffic Characteristics

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

- Traffic Volume;
- Design Vehicles; and
- Highway Capacity

4.3.1 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 projected annual average daily traffic (AADT) for the design year, 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.

4.3.1.1 Annual Average Daily Traffic

AADT represents the total traffic for a year divided by 365, or the annual average traffic volume per day. Due to seasonal, weekly, daily, or hourly variations, **AADT by itself lacks the level of precision to adequately predict the anticipated vehicular operations. Design Hourly Volume (DHV) is the preferred method for determining the anticipated vehicular operations of the roadway.**

4.3.1.2 Traffic Forecast

Historically, project-level traffic forecasts were developed and approved by TPP. However, at the time of the writing of this document, TPP is in the process of decentralizing these efforts.

Please consult the **TPP Traffic Forecast Analysis SOP** for additional details.

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.

Guidance on each of these methods or a combination of the methods at the project level for traffic forecasts can be found in the **TPP Traffic Forecast Analysis SOP**.

4.3.1.3 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 AADT, some adjustment of DHV may be appropriate.

4.3.1.4 Computation of DHV and DDHV

For one-way facilities, the AADT 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 $AADT_{NDIR}$ is the total traffic in both directions of travel (i.e., non-directional).

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

$$\mathbf{DHV} = (\mathbf{AADT_{NDIR}})(\mathbf{K})$$

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 **DES - Traffic Simulation and Safety Analysis Section webpage** and the **TSAP** 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 AADT.

The percentage of AADT 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 directional AADT to DHV as follows:

$$\mathbf{DHV} = (\mathbf{AADT})(\mathbf{K})(\mathbf{D})(\mathbf{2})$$

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

$$\mathbf{DDHV} = (\mathbf{AADT_{NDIR}})(\mathbf{K})(\mathbf{D})$$

4.3.1.5 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.

4.3.1.6 K Factor (K)

K is the percentage of AADT 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.

4.3.1.7 Service Flow Rate (SF)

A facility should be designed to provide sufficient capacity to accommodate the design

traffic volumes (AADT, 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 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 LOS procedures as detailed in the HCM.

4.3.2 Design Vehicles

The key controls in geometric highway design are the physical characteristics and the proportions of vehicles of various sizes using the roadway. Therefore, it is appropriate to examine all vehicle types, establish general class groupings, and select vehicles of representative sizes within each class for design use. These selected vehicles, with representative weight, dimensions, and operating characteristics, are used to establish roadway design controls for accommodating designated vehicle classes and are known as design vehicles.

There are four general classes of design vehicles:

- **Passenger Cars** include cars of all sizes, sport/utility vehicles, minivans, vans, and pickup trucks;
- **Buses** include motor coaches, city transit, school, and articulated buses;
- **Trucks** include single unit trucks, truck tractor semitrailer combinations, and truck tractors with semitrailers in combination with full trailers; and
- **Recreation Vehicles** include motor homes, cars with camper trailers, cars with boat trailers, and motor homes with pulling cars.

When designing the roadway facility, the designer should consider the largest design vehicle that is likely to use that facility with considerable frequency. **If a larger class of vehicle typically uses the facility, that vehicle shall govern the design, regardless of the minimum design vehicle required for the type of facility.**

A large school bus (84 passengers) should be utilized for the design of intersections of highways with low-volume county roads and township/local roads under 400 AADT along with some subdivision street intersections.

The WB-67 truck should generally be the minimum size design vehicle considered for intersections of freeway ramp terminals with arterial crossroads and for other intersections on State highways and industrialized streets that carry high volumes of truck traffic.

For more guidance on design vehicles and representations of design vehicles turning movements refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** and **NCHRP Report 505**.

Table 4-1 depicts the minimum turning radii for design vehicles.

4.3.3 Control Vehicles

Geometric design may also incorporate a control vehicle. The control vehicle is an infrequent but necessary user of the facility and informs what operations need to be accommodated in the design of the facility. The control vehicle is generally larger than a design vehicle. As an example, a facility may be designed for WB-67's but may also need to account for oversized tractor trailers carrying large materials such as wind turbines. Adopting both a design vehicle and a control vehicle encourages a safer design that can accommodate frequent and infrequent users. To prevent the overdesign of a facility, the control vehicle should only dictate how the design might accommodate a larger vehicle's needs when using the facility.

Typically, a design vehicle can turn using one incoming and one receiving lane, while the control vehicle can turn using multiple lane spaces. Engineering judgement should be exercised when determining how many lane spaces a control vehicle can use. **Figure 4-1** shows an example of how the turning movements can differ between a design vehicle and control vehicle.

Refer to **Section 13.10** and **Table 13-7** for additional information on minimum turning radii and edge of pavement designs for right turns to accommodate various design vehicles.

Table 4-1: Minimum Turning Radii of Design Vehicles

Design Vehicle Type	Passenger Car	Single-Unit Truck	Single-Unit Truck (Three Axle)	Intercity Bus (Motor Coach)		City Transit Bus	Conventional School Bus (65 pass.)	Large School Bus (84 pass.)[1]	Articulated Bus	Intermediate Semi-trailer
Symbol	**P**	**SU-30**	**SU-40**	**BUS-40**	**BUS-45**	**CITY-BUS**	**S-BUS36**	**S-BUS40**	**A-BUS**	**WB-40**
Minimum Design Turn Radius (ft)	23.8	41.8	51.2	41.7	44.0	41.6	38.6	39.1	39.4	39.9
Centerline Turning Radius (CTR) (ft)[4]	21.0	38.0	47.4	37.8	40.2	37.8	34.9	35.4	35.5	36.0
Minimum Inside Radius (ft)	14.4	28.4	36.4	24.3	24.7	24.5	23.8	25.3	21.3	19.3

Design Vehicle Type	Interstate Semi Trailer		"Double Bottom" Combination	Rocky Mtn Double	Triple Semi-trailer/ Trailers	Turnpike Double Semi- trailer / Trailer	Motor Home	Car and Camper Trailer	Car and Boat Trailer	Motor Home and Boat Trailer
Symbol	**WB-62[2] & WB-62TX**	**WB-67[3]**	**WB-67D**	**WB-92D**	**WB-100T**	**WB-109D[2]**	**MH**	**P/T**	**P/B**	**MH/B**
Minimum Design Turn Radius (ft)	44.8	44.8	44.8	82.0	44.8	59.9	39.7	32.9	23.8	49.8
Centerline Turning Radius (CTR) (ft)[4]	41.0	41.0	40.9	78.0	40.9	55.9	36.0	30.0	21.0	46.0
Minimum Inside Radius (ft)	7.4	1.9	19.1	55.6	9.7	13.8	26.0	18.3	8.0	35

Notes:

1. The turning radius assumed by a designer when investigating possible turning paths is a set of the centerline of the front axle of a vehicle. If the minimum turning path is assumed, the CTR approximately equals the minimum design turning radius minus one-half the front width of the vehicle.
2. Design vehicle with 48-ft trailer as adopted in 1982 Surface Transportation Assistance Act (STAA)
3. Design vehicle with 53-ft trailer as grandfathered in with 1982 Surface Transportation Assistance Act (STAA)
4. School buses are manufactured from 42-passenger to 84-passenger sizes. This corresponds to wheelbase lengths of 11.0- to 20.0-ft, respectively. For these different sizes, the minimum design turning radii vary from 28.1- to 39.1-ft and the minimum inside radii vary from 17.7- to 25.3-ft.
5. This table is not inclusive of all design vehicles. For specific design vehicles that are not included but are frequent users of a facility refer to NCHRP 505.

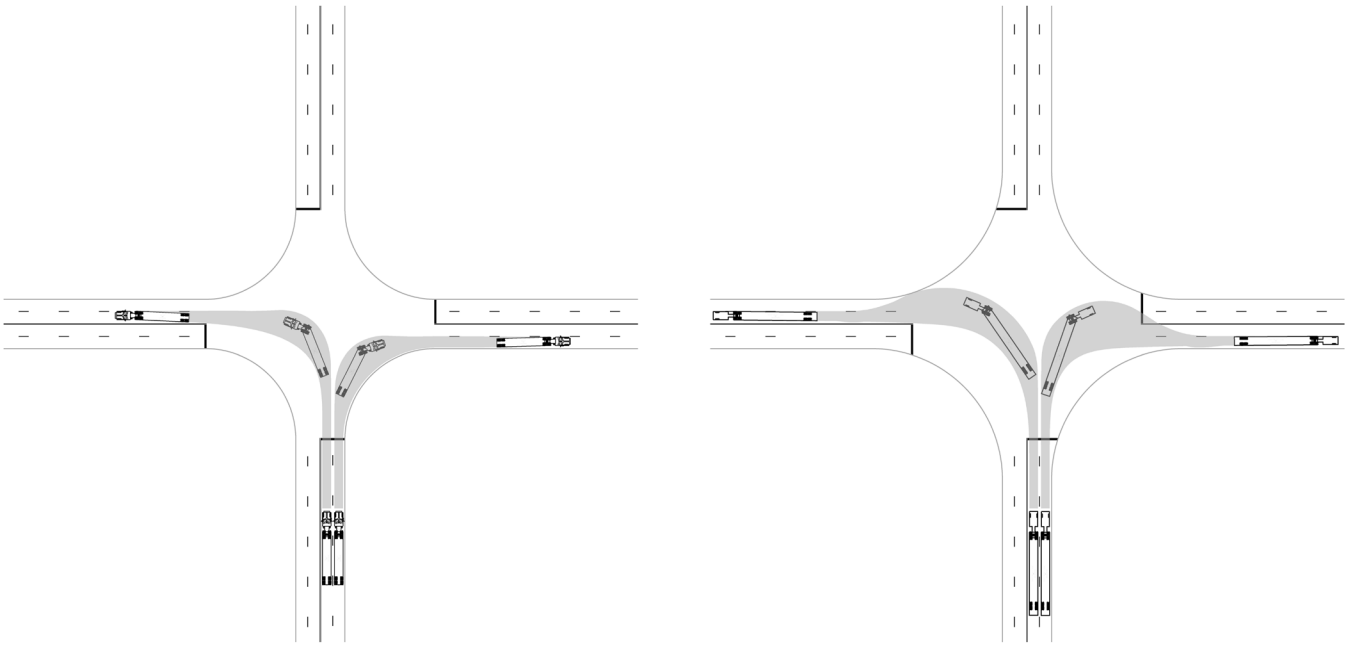


Figure 4-1: Lane usage for Design and Control Vehicle Turning Movements

4.3.4 Highway Capacity

Highway capacity is used to express the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a uniform section of a lane or roadway during a given period under roadway and traffic conditions. Highway capacity encompasses broader relations between highway characteristics and conditions, traffic composition and flow patterns, and the relative degree of congestion at various traffic volumes.

To determine the capacity for a particular roadway design, the designer should refer to the **HCM** for guidance. The **HCM** is used as the basic reference for performing operational analysis that may be appropriate depending on the circumstances and includes procedures for analyzing the LOS for all modes of travel.

Roadway capacity analysis serves three general purposes including:

- **Transportation planning studies**: used to assess the adequacy or sufficiency of existing roadway networks to service current traffic along with estimation of the future traffic growth that may exceed current capacity;
- **Roadway design**: used to select the roadway type and to determine dimensions such as the number and types of lanes and minimum lengths for weaving sections; and

- **Traffic operational analyses**: used to identify bottleneck locations of the roadway and to estimate operational improvements that may result from prospective traffic control measures or spot alterations in the roadway geometry.

While traffic volumes are the main contribution to the serviceability of a facility, roadway design features can also affect operating conditions. These roadway factors can be described as the following:

- **Alignment** for traffic traveling at any given speed, the better the roadway alignment the more traffic it can carry. It follows that congestions will generally be perceived at lower volumes if the design speed is low. The roadway should be subdivided into sections of consistent geometric design characteristics for analysis using the HCM techniques. A single limiting curve or steep grade in otherwise gentle alignment will be identified as the critical feature limiting roadway capacity.
- **Weaving Sections:** roadway segments where the pattern of traffic entering and leaving at contiguous points of access results in vehicle paths crossing each other. Where the distance in which the crossing is accomplished is relatively short in relation to the volume of weaving traffic, operations within the roadway section will be congested. Some reduction in operating efficiency through weaving sections can be tolerated by roadway users if the reduction is minor and the occurrence is not high.
- **Ramp Terminals:** these features can adversely influence operating conditions of freeways if the demand for their use is excessive or if their design is deficient. When congestion occurs at the freeway ramp junctions, typically drivers will avoid this lane and shift to the inside lanes causing congestion to the through lanes. The loss in efficiency is a function of the volume of traffic entering or leaving ramps, and the geometric layout of the terminals.
- **Mixture of Vehicles:** traffic streams are usually composed of a mixture of vehicles including passenger cars, trucks, buses, and occasionally recreational vehicles and bicycles. The combination of these vehicles creates a flow rate that is not uniform throughout the hour, day, season, or year.
- **Peak Hour Factor:** the factor used to convert the rate of flow during the highest 15-minute period to the total hourly volume is the peak hour factor (PHF). It is the ratio of the total hourly volume to the number of vehicles during the highest 15-minute period multiplied by 4.

4.3.4.1 Level of Service (LOS)

The HCM defines the quality of traffic service provided by specific roadway facilities under specific traffic demands by means of a LOS. The LOS characterizes the operating conditions on the facility in terms of traffic performance measures related to speed and travel time, freedom to maneuver, traffic interruptions, and comfort and convenience. The levels of service range from LOS A (least congested) to LOS F (most congested). The HCM procedures for estimating the traffic operation LOS should be used for all contexts. Designers should provide the highest LOS practical and consistent with the project functional and context classification. **Chapters 5 through 8** discuss the target LOS based on functional class.

Table 4-2: Level of Service

Level of Service	General Operating Conditions
A	Free Flow
B	Reasonably Free Flow
C	Stable Flow
D	Approaching Unstable Flow
E	Unstable Flow
F	Forced Breakdown Flow

4.4 Access Management

Access management assists in protecting the substantial public investment in transportation by preserving roadway efficiency and enhancing traffic safety. Access management can significantly reduce traffic accidents, personal injury, and property damage. **The State may regulate driveway location and design through its police powers.** Landlocking, however, through complete denial of access is beyond the State's regulatory power. 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.

See **Chapter 8** for more in-depth discussion of access control along mainlanes, frontage roads, and methods of access control for freeways.

For guidance of placement of driveways and land access, refer to the **AMM** for additional information on the determination of control of access limits.

4.5 Safety

Refer to the **TSAP** for a detailed overview of TxDOT's traffic and safety analysis procedures.

For Safety Analyses of 3R projects, see **Chapter 10**.

4.6 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 predominant 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.

4.6.1 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 percent to 8 percent.**

4.6.2 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 percent to 15 percent.**

4.6.3 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 percent.**

4.7 Horizontal Alignment

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.

4.7.1 General Considerations

There are several 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 except where very unusual topographical or ROW conditions make other alternatives impractical. This configuration is unexpected by drivers, not pleasing in appearance, and more difficult for freight truck maneuverability. **It is recommended to provide a minimum tangent length that is at least 15 times the design speed.** A design exception is not required if this recommended tangent length is not met.
- Horizontal alignment and its associated design speed should be consistent with other design features and topography. Combination of horizontal and vertical alignments is discussed in **Section 4.9**.

4.7.2 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 rural town, suburban, urban, and urban core facilities (45-mph and below) is shown in **Table 4-4**. Design guidance for curvature of high-speed (50-mph and above) or rural context facilities is shown in **Table 4-5**, **Table 4-6** and **Table 4-7** for maximum superelevation (e_{max}) rates equal to 4 percent, 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.

4.7.3 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:

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

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.

To provide designers flexibility in high-speed urban and suburban settings, a maximum superelevation rate of 4 percent may be used instead of a 6 percent or 8 percent superelevation rate. **Freeway facilities are excluded from using a maximum superelevation rate of 4 percent.**

4.7.3.1 Methods of Calculating Superelevation Rate

When calculating superelevation on rural and high-speed suburban, urban, and urban core facilities, AASHTO uses a process known as **Method 5**. Method 5 is intended to accommodate overdriving that is likely to occur on flat to intermediate curves. Overdriving on such curves involves little risk that a driver will lose control of the vehicle because superelevation sustains nearly all the lateral acceleration at the average running speed and

a large amount of side friction is available for greater speeds. Due to the additional side friction available, Method 5 is used on rural and high-speed suburban, urban, and urban core facilities to provide greater driver comfort and safety.

Where superelevation will be applied to low-speed roads on low-speed rural town, suburban, urban, and urban core facilities, AASHTO uses **Method 2** superelevation distribution. Method 2 only introduces superelevation after the maximum side friction has been used. Therefore, no superelevation is needed on flatter curves. This method is used on low-speed rural town, suburban, urban, and urban core facilities where, because of various constraints, superelevation frequently cannot be provided. Method 2 ensures driver safety but does not offer the added driver comfort that Method 5 provides.

Table 4-3 provides a summary of scenarios for the use of Method 2 and Method 5. The following assumptions are included in the application of **Table 4-3.**

- **Urban context** includes Urban Core, Urban, Suburban, and Rural Town.
- **Rural context** is exclusive of urbanized contexts;
- **Intermediate Speed** is a range of speeds from 50 – 60 mph that technically falls within what would typically be categorized as a High-speed facility, the Intermediate values are provided in **Table 4-3** to give the designer additional flexibility in these transitional areas;
- The general continuum for the amount of calculated side force on a vehicle in a horizontal curve from greatest to least is as follows:
    - Method 2;
    - Method 5 (4 percent e_{max});
    - Method 5 (6 percent e_{max}); and
    - Method 5 (8 percent e_{max}).

  The designer has the option of selecting a superelevation methodology that either increases the radius and/or decreases the side force for driver comfort on the various facility types listed **Table 4-3**; and
- Side force should never exceed AASHTO maximums for a given design speed, regardless of the method used.

4.7.4 Superelevation Rates on Low-Speed Rural Town, Suburban, Urban and Urban Core Facilities

Although superelevation is advantageous for traffic operations, various factors often combine to make its use impractical in many rural town, suburban, urban, and urban core 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 rural town, suburban, urban, and urban

core 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 rural town, suburban, urban, and urban core facilities should be designed using NC, such that superelevation is not necessary where practical. This is accomplished by using the negative e-values from **Table 4-4**.

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 4-4**.

Table 4-3: Superelevation Methodology Summary[1]

	Low-Speed (≤ 45 mph)	Intermediate-Speed (50 – 60 mph)	High-Speed (65 mph and greater)
Urban All Functional Classifications (Excluding Freeway Mainlanes, Ramps and Direct Connectors)	Method 2 Table 4-4	Method 5 4%, 6% or 8% emax Table 4-5, Table 4-6 or Table 4-7	Method 5 6% or 8% emax Table 4-6 or Table 4-7
Rural All Functional Classifications	Method 5 6% or 8% emax Table 4-6, or Table 4-7		
Urban or Rural Freeway Mainlanes, Ramps, and Direct Connectors	Method 5 6% or 8% emax Table 4-6 or Table 4-7		
Urban Frontage Roads	Method 2 Table 4-4	Method 5 4%, 6% or 8% emax Table 4-5, Table 4-6 or Table 4-7	Method 5 6% or 8% emax Table 4-6 or Table 4-7
Rural Frontage Roads	Method 5 6% or 8% emax Table 4-6 or Table 4-7		
Urban Ramps for Grade Separations on Non-Access Controlled Facilities	Method 2 Table 4 4	Method 5 4%, 6% or 8% emax Table 4-5, Table 4-6 or Table 4-7	Method 5 6% or 8% emax Table 4-6 or Table 4-7
Rural Ramps for Grade Separations on Non-Access Controlled Facilities	Method 5 6% or 8% emax Table 4 6 or Table 4 7		
Roundabouts and Alternative Intersections (Including Approaches)[2]	Method 2 Table 4-4	Method 5 4%, 6% or 8% emax Table 4-5, Table 4-6 or Table 4-7	N/A
Temporary Traffic Control[3]	Method 2 expanded Table 23-1		
Low-Volume Off-System Bridges (approach roadway)	Meet or improve conditions that are typical on the remainder of the roadway.		

Notes:

1. The designer has the option of selecting a superelevation methodology that either increases the radius and or decreases the side force for driver comfort on the various facility types listed in this table.
2. The desired target speed approaching and through roundabouts and other alternative intersection forms should be lower to ensure the proper functioning of the alternative intersection and the resultant safety benefits.
3. For areas in the TCP phasing of construction that represent what will be left in place for the final permanent condition, the respective permanent condition superelevation method should be utilized.

Table 4-4: Superelevation Rates on Low-speed Rural Town, Suburban, Urban and Urban Core Facilities (Method 2)

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	1227
-3.0[2]	52	111	208	353	544	821	1125
-2.8[2]	51	110	206	349	537	808	1107
-2.6[2]	51	109	204	345	530	796	1089
-2.5[2,3]	51	109	203	343	527	790	1080
-2.4[2]	51	108	202	341	524	784	1071
-2.2[2]	50	108	200	337	517	773	1055
-2.0	50	107	198	333	510	762	1039
-1.5[4,5]	49	105	194	324	495	736	1000
-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.

Table 4-4, which is based on the Method 2 superelevation distribution, shows the relationship of radius, superelevation rate, and design speed for low-speed rural town, suburban, urban, and urban core facility design and should be used to evaluate existing conditions or the need for superelevation for proposed conditions on low-speed rural town, suburban, urban, and urban core facilities. **This table may also be used for design of detour alignments in constrained conditions.**

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 35 mph and a 400-ft radius curve, Table 4-4 indicates an approximate superelevation rate of 2.4 percent should be used.

4.7.5 Superelevation Rate on Rural and High-Speed Suburban, Urban, and Urban Core Facilities

Table 4-5, Table 4-6 and Table 4-7 show superelevation rates (maximum 4 percent, 6 percent and 8 percent, respectively) for various design speeds and radii based on **Method 5** superelevation distribution.

These tables should be used for rural and high-speed suburban, urban, and urban core facilities. **For multi-lane facilities, particularly where wide medians are used, the radius applies to the inside edge of the innermost travel lane.**

Table 4-5: Minimum Radii and Superelevation Rates[1] for High-Speed Suburban and Urban Non-Freeway Facilties, e_{max} = 4%[1, 2] (Method 5)

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)
NC[3,5]	See Table 4-4 (Method 2)							7,220	8,650	10,300
RC[4,5]								4,940	5,950	7,080
2.2								4,280	5,180	6,190
2.4								3,690	4,500	5,410
2.6								3,130	3,870	4,700
2.8								2,660	3,310	4,060
3.0								2,290	2,860	3,530
3.2								1,980	2,490	3,090
3.4								1,720	2,170	2,700
3.6								1,480	1,880	2,350
3.8								1,260	1,600	2,010
4.0								926	1,190	1,500

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. Use of e_{max}=4% should be limited to high-speed urban and suburban areas. Freeway facilities are excluded from using e_{max}=4%.
3. 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.
4. 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.
5. 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 4-6: Minimum Radii and Superelevation Rates[1] for Rural and High-Speed Suburban, Urban and-Urban Core 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[2,4]	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
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. 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, rather than interpolation a superelevation rate equal to the normal crown should typically be used.
5. Low-Speed values are shown for use on Rural Facilities with low operating speeds. These values are also provided for design exception documentation on High-Speed facilities.

Table 4-7: Minimum Radii and Superelevation Rates[1] for Rural and High-Speed Suburban, Urban and Urban Core 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	959	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
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

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 super elevating 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.

4.7.6 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**.

Preferable 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 4-8** 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 4-8: 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
Notes:		
1. Maximum relative gradient for profile between edge of traveled way and axis of rotation.		

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

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

Where:

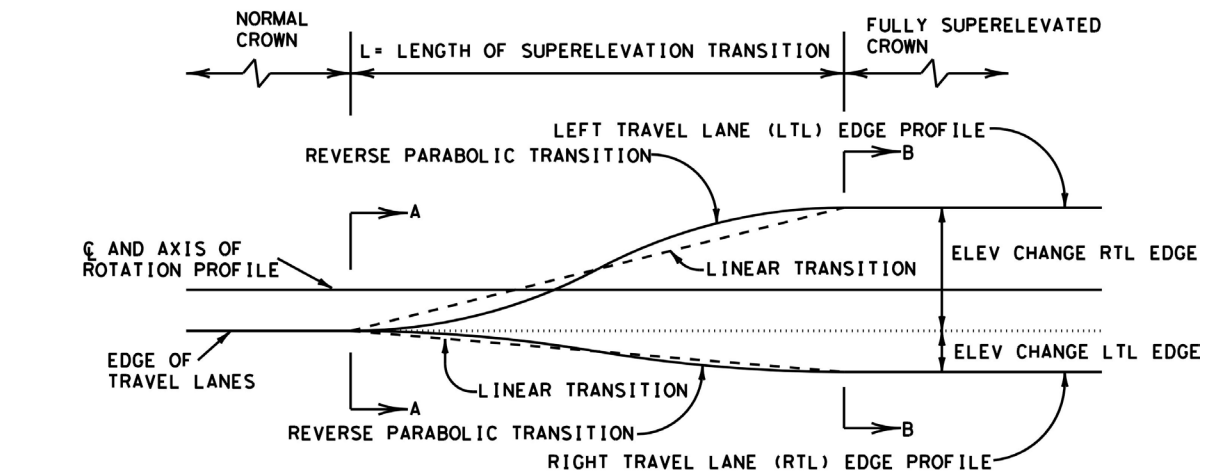
$\mathbf{L_{CT\ (preferable)}}$ = Calculated preferable transition length, ft

CS = Change in cross slope of superelevated pavement, percent

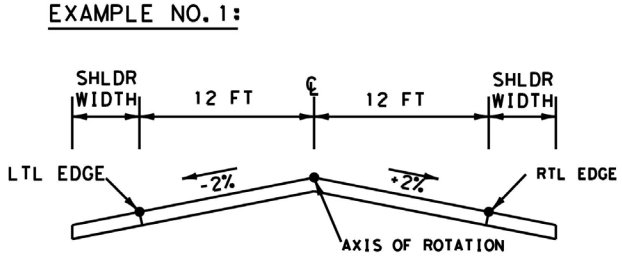
W = distance between the axis of rotation and the edge of traveled way, ft

G = maximum relative gradient (%).

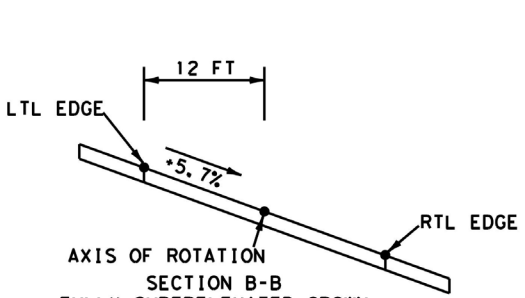
Example determinations of superelevation transition are shown in **Figure 4-2**.



EXAMPLE NO. 1:



SECTION A-A
NORMAL CROWN



SECTION B-B
FULLY SUPERELEVATED CROWN

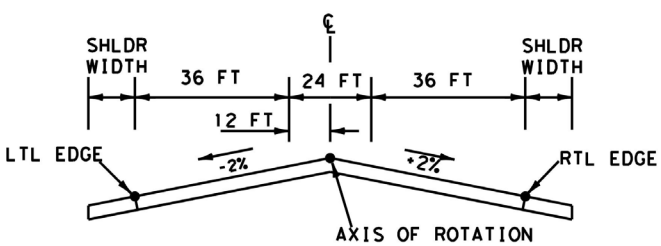
GIVEN: RURAL TWO LANE HIGHWAY
DESIGN SPEED=60 mph
R=2500 FT, TO THE RIGHT
℄ IS ROTATION AXIS
MAX. e=8%
REQUIRED e=5.7%

FIND: LENGTH OF SUPERELEVATION TRANSITION

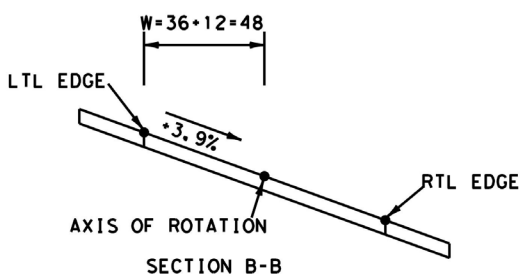
SOLUTION

CALCULATE L_{CT} : $L_{CT} = (CS)(W)/(G)$
MAX REL G=0.50 (FROM TABLE 2-6)
L_{CT} = [-2-(+5.7)] [12] / [0.50]
L_{CT} = 185 FT

EXAMPLE NO. 2:



SECTION A-A
NORMAL CROWN



SECTION B-B
FULLY SUPERELEVATED CROWN

GIVEN: URBAN SIX-LANE FREEWAY
DESIGN SPEED=60 mph
R=4000 FT, TO THE RIGHT
℄ IS ROTATION AXIS
MAX. e=8%
REQUIRED e=3.9%

FIND: LENGTH OF SUPERELEVATION TRANSITION

SOLUTION

CALCULATE L_{CT} : $L_{CT} = (CS)(W)/(G)$
MAX REL G=0.50 (FROM TABLE 2-6)
L_{CT} = [-2-(+3.9)] [48] / [0.50]
L_{CT} = 567 FT

Figure 4-2: 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 preferable, the length for multilane 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 4-9**, such that the transition length formula becomes:

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

where "**b**" is defined in Table 4-9.

In the case of one lane being rotated, "b" is 1.0, such that $L_{CT\ (min)} = L_{CT\ (preferable)}$

Table 4-9: Multilane Adjustment Factor[1]

Number of Lanes Rotated (n)	Adjustment Factor[2] (b)
1.5	0.83
2	0.75
2.5	0.70
3	0.67
3.5	0.64
4	0.63
4.5	0.61
5	0.60

Notes:

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.
2. For all values of n, $b = \frac{1+0.5(n-1)}{n}$ (Calculations are rounded to the nearest hundredth)

4.7.7 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 4-10. 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 4-10: 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

Notes:

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 4-5** 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. Preferably, **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 superelevation transition section.** At a minimum, either criterion should be met.

On existing alignments, 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.

4.7.8 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 4-2 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.

4.7.9 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 4-23**), 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 (2D or 3D) 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 4-3** illustrates a graphical approach to determining horizontal sight distance in a curve.

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:

- Increasing the offset to the obstruction; or
- Increasing the radius.

However, the alternative should not incorporate a shoulder width on the inside of the curve more than 12-ft because of the concern that drivers will use wider shoulders as a passing or travel lane.

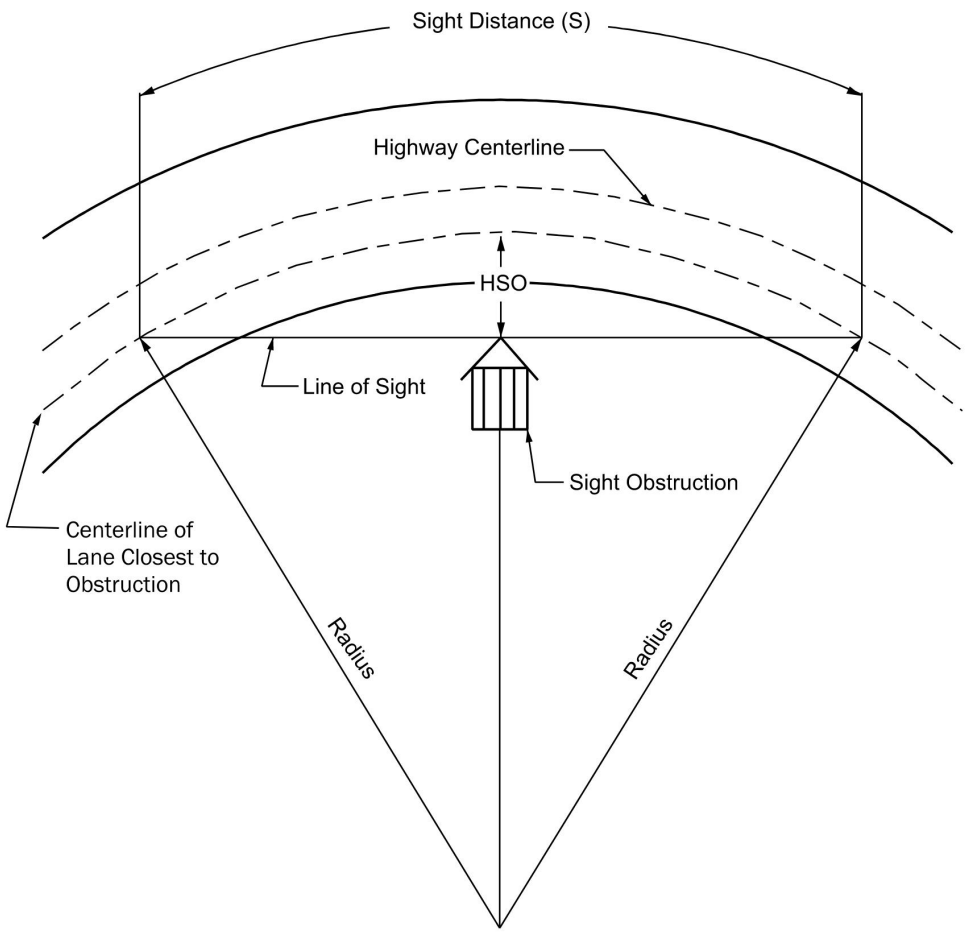


Figure 4-3: Diagram Illustrating Components for Determining Horizontal Sight Distance

Source: AASHTO A Policy on Geometric Design of Highways and Streets

4.8 Vertical Alignment

The two basic elements of vertical alignment are:

- **Grades; and**
- **Vertical curves.**

4.8.1 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 4-4** 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. For additional entry speeds and/or design vehicles see **AASHTO's A Policy on Geometric Design of Highways and Streets**.

Where critical length of grade is exceeded for any type of two-lane facility, climbing lanes should be considered as discussed in the HCM.

When the critical length of grade values shown in **Figure 4-4** 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).

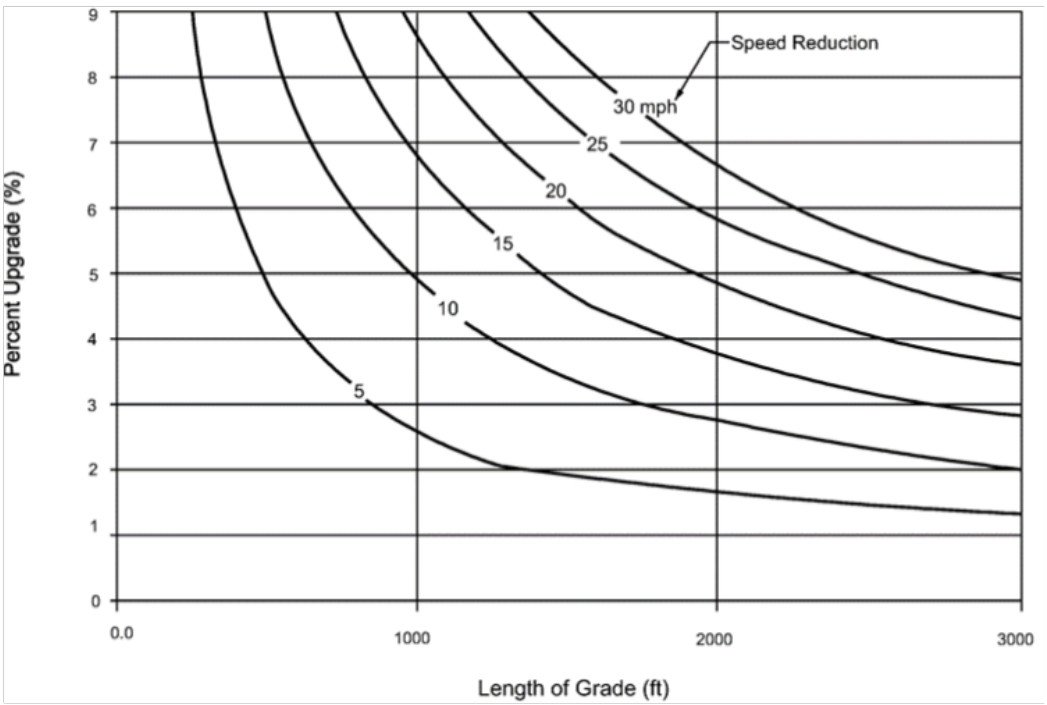


Figure 4-4: Critical Lengths of Grade for Design, Assumed Typical Heavy Truck of 200-lb/hp, Entering Speed = 70 mph

Source: AASHTO A Policy on Geometric Design of Highways and Streets

Table 4-11: Maximum Grades[2]

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

Notes:

1. Up to 15% maximum can be used in very constrained conditions. Flatter gradients should be used where practical and grades less than 5% are Preferable.
2. The maximum grades shown may be increased by 1% where there are one-way downgrades that are less than 500-ft in length. In these cases, a design exception would not be required.
3. Where heavy truck traffic is present (over 10%) it is recommended to use a max grade of 5% in level terrain and 6% in rolling terrain.
4. See 15.7.3 for ramp and direct connector criteria.

Table 4-11 summarizes the maximum grade controls in terms of design speed. See **Section 4.6** for defining level, rolling, and mountainous terrain conditions. Generally, the **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.

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 Section 4.9). **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 TxDOT's Hydraulic Design Manual (HDM).

4.8.2 Vertical Curves

Vertical curves provide gradual changes between tangents of different grades. The simple parabola shown in Figure 4-5 is used in the highway profile design of vertical curves.

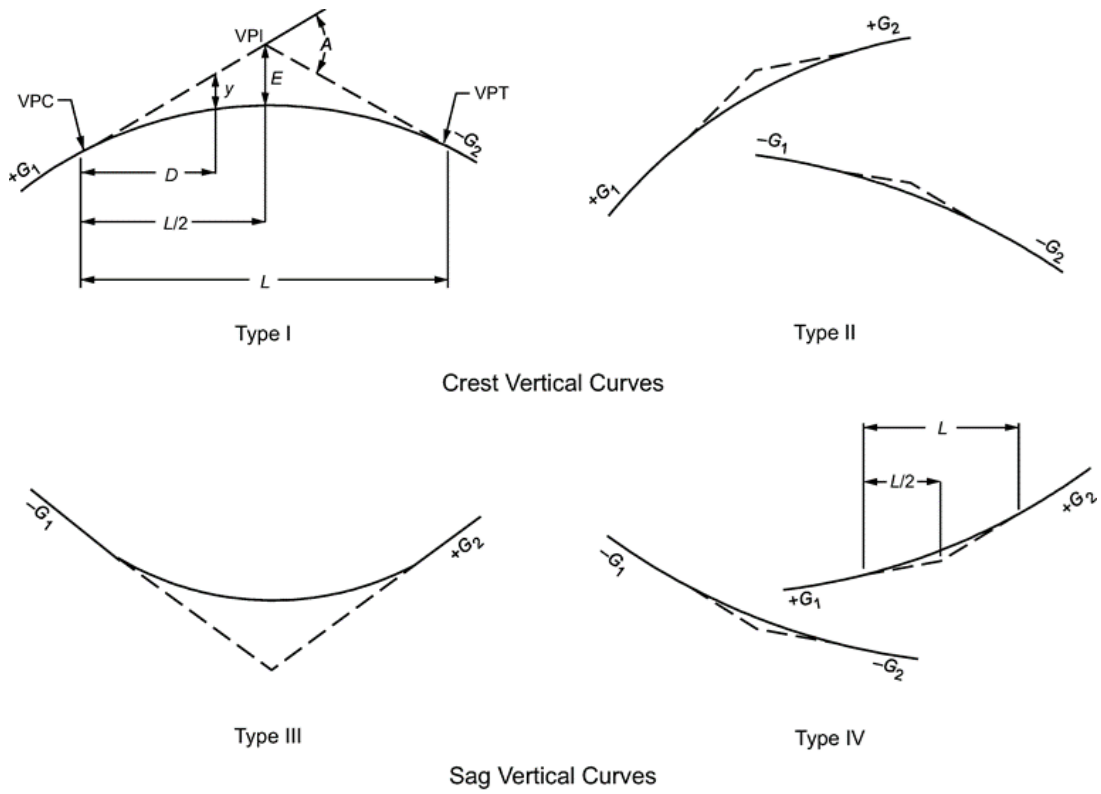


Figure 4-5: Types of Vertical Curves

Source: AASHTO A Policy on Geometric Design of Highways and Streets

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

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

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

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

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 ($G_1 - G_2$), 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 4-6** and **Figure 4-7**, respectively. The solid lines give the minimum vertical curve lengths on the basis of rounded values of K. These lengths represent minimum K-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 in **Section 4.8.3**), a minimum length of vertical curve is shown. Attention should be given where there are successive vertical curves.

Preferable minimum lengths of vertical curves (both crest and sag) are three times the design speed.

$$L_{min} = 3*DS$$

This minimum length is not considered a design control (i.e., a design exception, 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:

- Headlight sight distance;
- Passenger comfort;
- Drainage control; and
- 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 4-7** 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. This level point will only occur on Type I and Type III vertical curves shown in **Figure 4-5**. 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 4-6** and **Figure 4-7** 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.

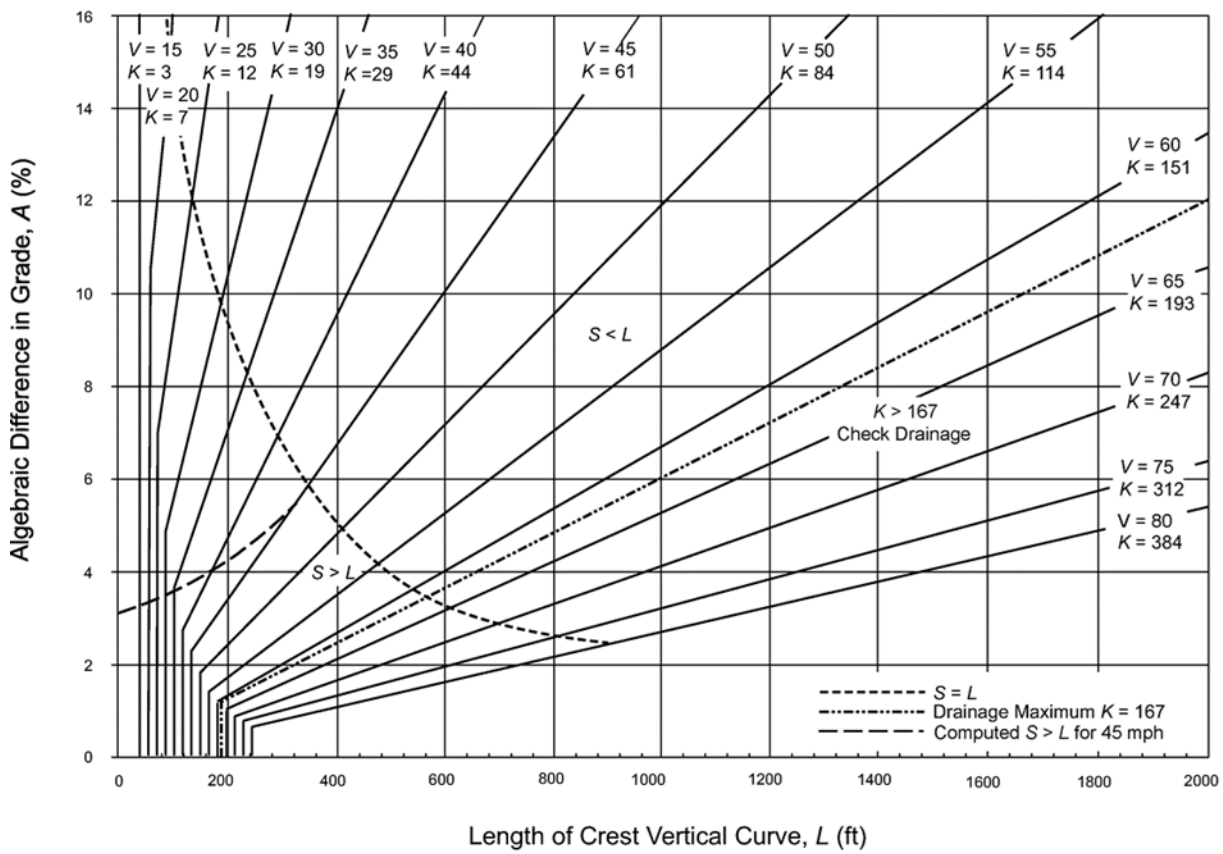


Figure 4-6: Design Controls for Crest Vertical Curves

Source: AASHTO A Policy on Geometric Design of Highways and Streets

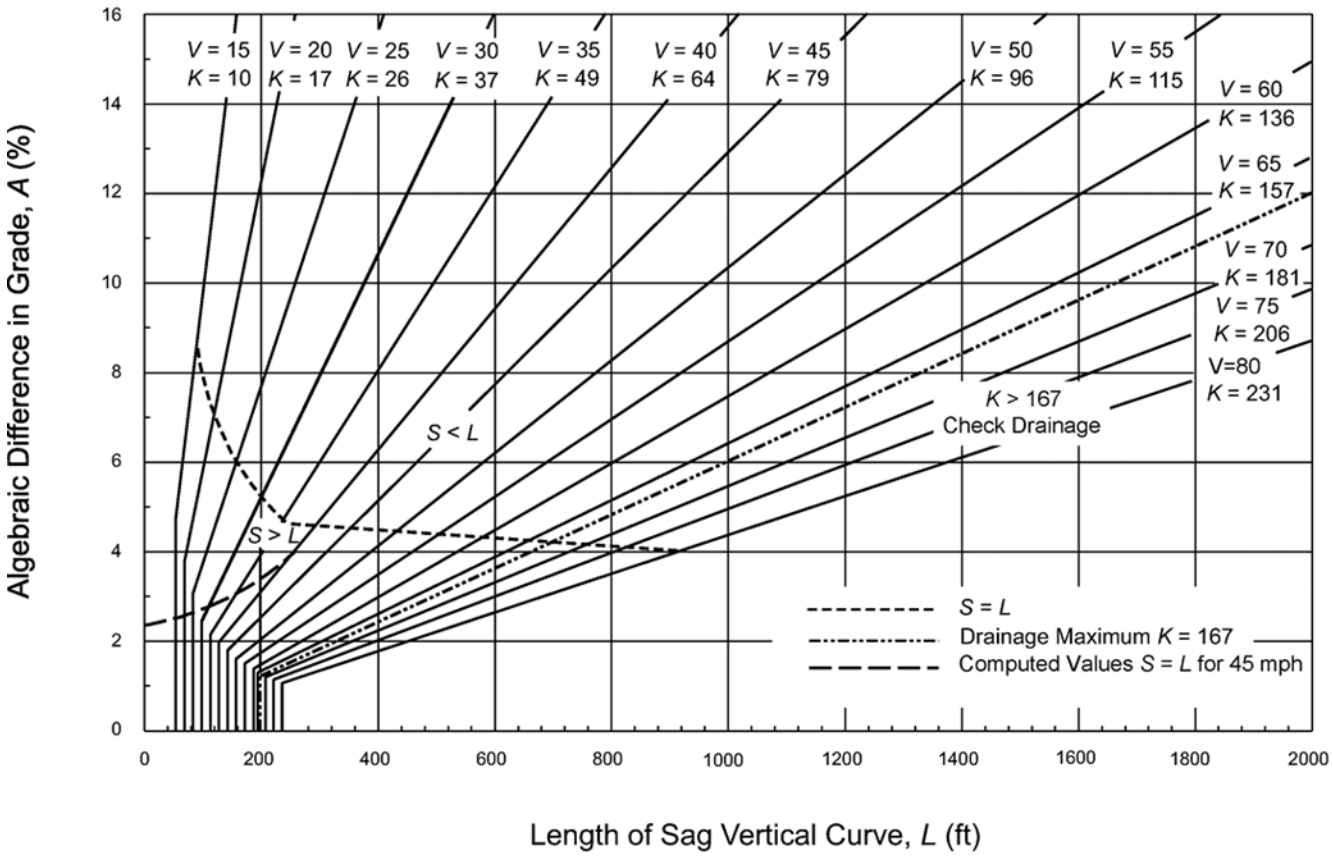


Figure 4-7: Design Controls for Sag Vertical Curves

Source: AASHTO A Policy on Geometric Design of Highways and Streets

Table 4-12 provides minimum vertical curve lengths and associated K as a function of A. **Longer curves should be used wherever practical.**

Table 4-12: Minimum K and L as a Function of A

Design Speed (mph)	Crest Vertical Curves			Sag Vertical Curves		
	A (%)	Min K	Preferable Length of Curve[2] (ft)	A (%)	Min K	Preferable Length of Curve[2] (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
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

Notes:

1. Exceeds recommended drainage maximum of K > 167. Ensure proper pavement drainage.
2. This length is not considered a design control (i.e., a design exception, design waiver would not be required for these length values if the minimum stopping sight distance for the relevant design speed is met).

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.

See **Figure 4-29** 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.

When vertical curves are located within the limits of a bridge, it is preferred to locate the VPI, VPC, VPT's in the center of the bridge abutment(s) or bent(s) to simplify construction.

4.8.3 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). It is recommended to always use vertical curves to address grade changes for the following conditions.

- 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 (the approach with the lower ADT) 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 for the minor approach of an intersection with yield or stop control.

Use of this criterion should be coordinated with the District Design Engineer prior to implementation and documented in the project files.

4.8.4 Vertical Alignment at Railroad Crossings

It is desirable that the intersection of highway and railroad be as level as possible from the standpoint 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 TxDOT's **Rail Highway Operations Manual** (Chapter 7, Section 1) and **AASHTO's A Policy on Geometric Design for Highways and Streets** for additional information.

4.8.5 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 an intersection with stop or yield 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 all other intersections.
    - 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. See **Chapter 19** for additional guidance and when TDLR variances are required.

It is generally preferable 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** (the roadway with the higher ADT) 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 percent should be maintained along corner radii of curbed intersections.

The combination of vertical and horizontal alignment should allow adequate sight distance at an intersection. A sharp horizontal curve following a crest vertical curve is undesirable, particularly on intersection approaches.

4.8.6 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. This includes 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 4-13** shows the minimum vertical clearances required. **Section 21.2** provides additional guidance for roadways on the THFN.

See Chapter 1 for additional information on vertical clearance design exceptions, and THFN design deviations.

During construction, refer to TxDOT's **Bridge Design Manual - LRFD** for requirements on maximum time between erection of girders and placement of deck for spans overactive lanes of traffic 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.

A clearance of 15-ft is preferable to provide a sufficient buffer between allowed and actual heights to avoid over-height impacts.

Table 4-13: Vertical Clearance Requirements[1]

Functional Classification	Vehicle Overpasses[2] (ft)	Overhead Sign Structures and Bicycle / Pedestrian Overpasses[4] (ft)
Local/Collector/Arterial	16.5[3]	17.5
Freeway	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. Includes overhead signs, light fixtures, sign bridges, and toll gantries.
5. If an overlay reduces the vertical clearance below 18.0-ft, a THFN Design Deviation will be required.

4.9 Combination of Horizontal and Vertical 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; and
- 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; and
- 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.

For rural two-lane 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.

4.10 Cross Sectional Elements

4.10.1 Through Travel Lanes

Subsequent chapters of this manual identify appropriate lane widths for the various functional and context classifications.

4.10.2 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 as presented in Appendix B.**

Speed change lanes may be provided as space for deceleration/acceleration to/from intersecting side streets with significant volumes and high operating speeds. They also provide space for deceleration and optional storage of turning vehicles. The length of speed change lanes for turning vehicles consists of the following to components:

- Deceleration length; and
- Storage length.

Table 4-14: Lengths of Left, Right, Median, and Dual Turn Lanes[1]

Design Speed (mph)	Deceleration Length (ft) [2] Speed Differential [3]			Taper Length (ft)	Minimum [6,7] Storage Length (ft)
	None	5-mph[4]	10-mph[5]		
30	150	105	70	50[9]	100
35	205	150	105	50[9]	
40	265	205	150	50[9]	
45	340	265	205	100[8]	
50	415	340	265	100[8]	
55	505	415	340	100[8]	
60	600	505	415	100[8]	
65	700	605	515	150	
70	815	720	630	150	
75	935	840	750	150	
80	1060	965	875	150	

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/s^2 deceleration to stopped condition throughout the entire length. Larger deceleration rates, up to 9.9-ft/s^2, may be used when deceleration lengths based on 6.5 ft/s^2 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/s^2 deceleration from 5 mph less than design speed to stopped condition throughout the entire length.
5. Based on 6.5 ft/s^2 deceleration from 10 mph less than design speed to stopped condition throughout the entire length.
6. See Storage Length Calculations discussion.
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 would then be required if the 100 ft minimum storage length (30 ft minimum for right-turn lanes) cannot be provided.
8. For dual left turn and right turn lanes 150-ft taper to be used.
9. For dual left turn and right turn lanes 100-ft taper to be used.

4.10.2.1 Width of Speed Change Lanes

The preferable width of a speed change lane should match the through lane width of the facility. In constrained situations the speed change lane width may be reduced to 11- or 10-ft on arterial and collector facilities. A design waver will be required if a width of 10-ft is not met except on local roadways. Local roadways will require a design waiver if a width of 9-ft is not provided.

4.10.2.2 Left-Turn Deceleration Lanes

Figure 4-13 illustrates the use of left-turn lanes for all contexts. 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, if applicable) is recommended. Where pedestrian refuge is needed, a median divider width of 6-ft is required (measured minimum to face of curb, preferable 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 19** for additional guidance. **Table 4-14** 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.

For Rural roadways, adjustments to the deceleration length should be applied to a turn lane if the longitudinal grade exceeds 3 percent. See **Table 4-19** for adjustments factors due to steep longitudinal grades.

4.10.2.2.1 Rural Left-Turn Lane Warrants

The additional expense of adding left-turn lanes on rural two-lane highways at intersecting crossroads is often not justified due to low volumes. **Figure 4-8** provides recommendations for when left-turn lanes should be considered for a typical rural two-lane highway intersection. Left-turn lanes may be added based recent crash data or projected volumes associated with planned development.

In instances on three-leg rural 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 4-8** and **Figure 4-9**.

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 4-9** shows typical geometry for a rural two-lane highway with left-turn bays at an intersecting crossroad.

Table 4-15: 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 or More	< 50	50	< 50
Notes:			
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 4-8			

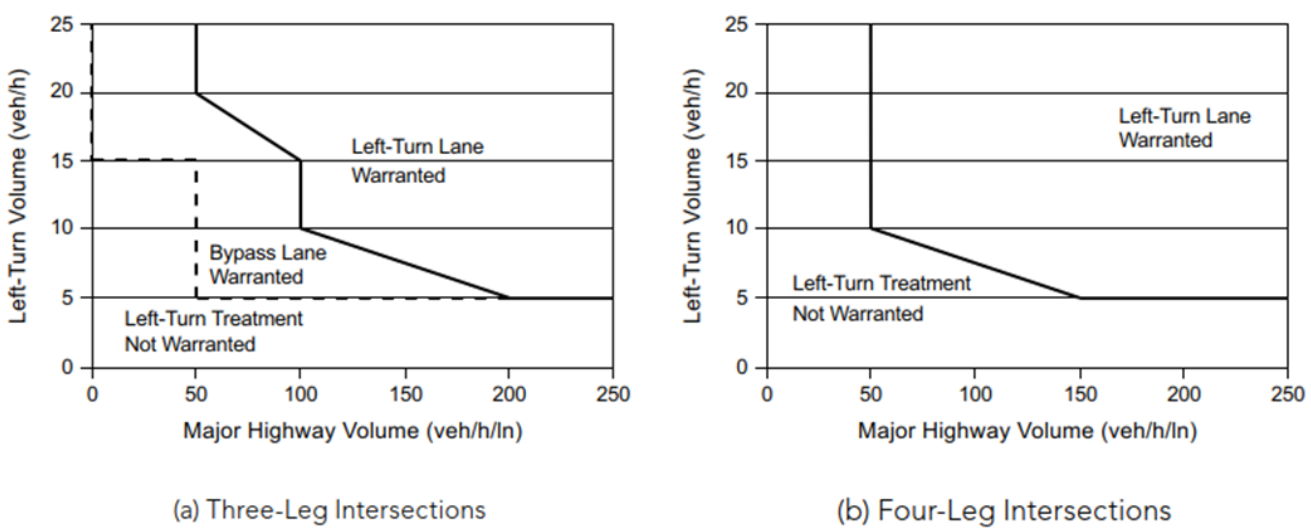
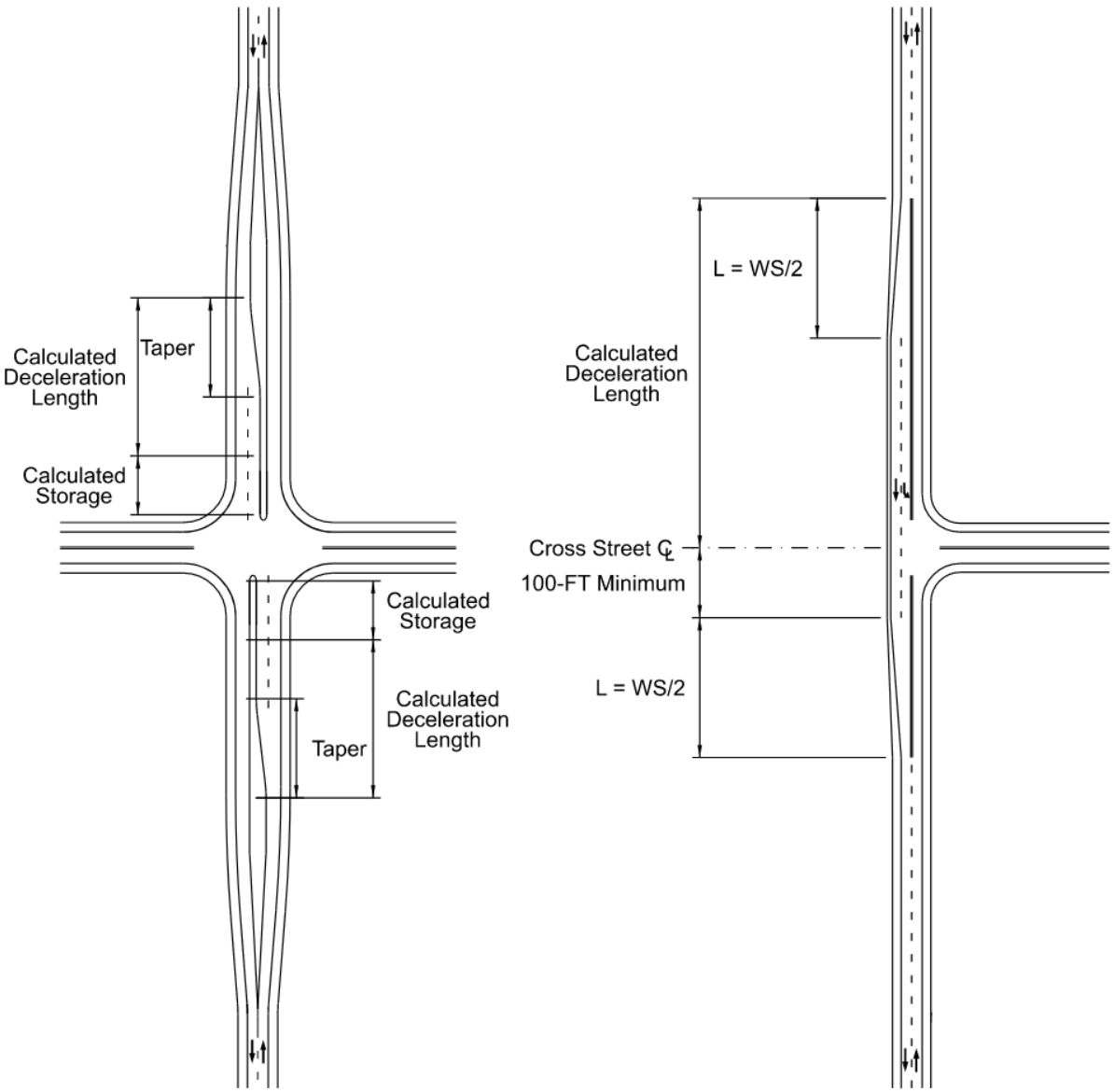


Figure 4-8: Suggested Left-Turn Warrants Based on Results from Benefit-Cost Evaluations for Intersections on Two-Lane Highways in Rural Areas

Source: AASHTO A Policy on Geometric Design of Highways and Streets



Intersection With Left Turn Lanes (A)

T-Intersection with Left-Turn Bypass Lane (B)

Notes:

1. This is not intended to show striping or pavement marking details. Refer to the Texas MUTCD for additional information.
2. See Chapter 13, Intersections for additional design guidance on turn lanes.
3. See Section 4.10.5 for additional design guidance on tapers.
4. These figures are not intended to convey lane or shoulder widths. It is preferred to match the approach shoulder widths throughout a left-turn by-pass lane. If the shoulder width needs to be reduced for constrained situations, then it may be reduced to the minimum width for the appropriate functional classification shown in Appendix A.

Figure 4-9: Rural Two-Lane Highway with Left-Turn Bays at Intersecting Crossroad

Suggested guidelines and warrants for the installation of left-turn lanes on four-lane rural highways based on turning and through volumes is provided in **Table 4-16** and **Figure 4-10.**

These volume-based guidelines indicate situations where a left-turn lane may be desirable, not necessarily situations where a left-turn lane is required. Further discussion and examples of left-turn lane guidance can be found in **AASHTO's Policy on Geometric Design of Highways and Streets**.

Table 4-16: Guide for Left-Turn Lane Warrants for Four-Lane Highways in Rural Areas

Source: AASHTO A Policy on Geometric Design of Highways and Streets

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	75	50
10	75	25
15	50	25
20	50	25
25	50	< 25
30	50	< 25
35	50	< 25
40	50	< 25
45	50	< 25
50 or More	50	< 25
Notes:		
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 4-10.		

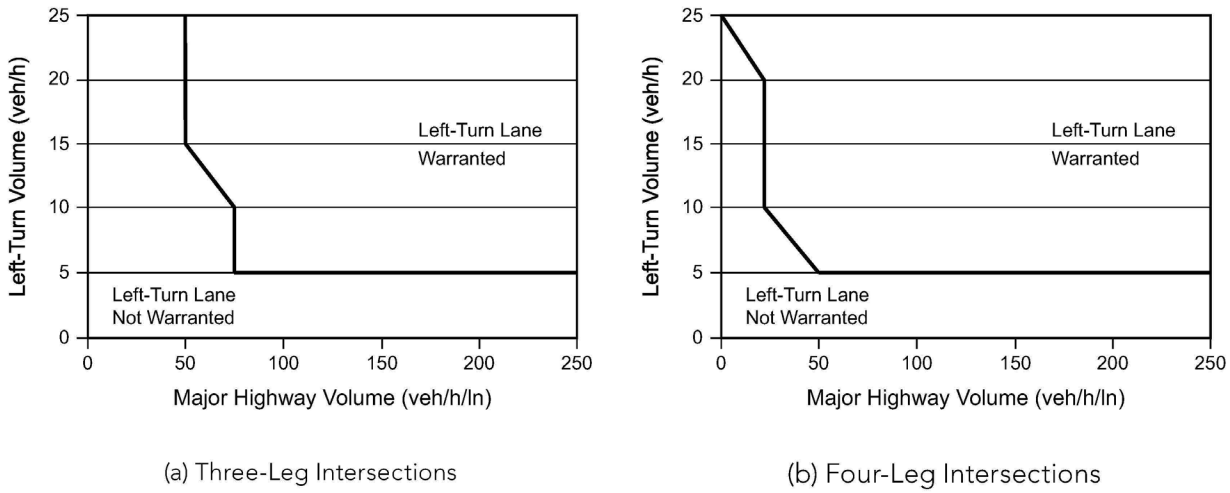


Figure 4-10: Suggested Left-Turn Warrants Based on Results from Benefit-Cost Evaluations for Rural Four-Lane Highways

Source: AASHTO A Policy on Geometric Design of Highways and Streets

4.10.2.2.2 Rural Town, Suburban, Urban, and Urban Core Left-Turn Lane Warrants

Suggested guidelines and warrants for the installation of left-turn lanes in urban and suburban areas based on turning and through volumes is provided in **Table 4-17** and **Figure 4-11**. **These volume-based guidelines indicate situations where a left-turn lane may be desirable, not necessarily situations where a left-turn lane is required**. Further discussion and examples of left-turn lane guidance can be found in **AASHTO's Policy on Geometric Design of Highways and Streets**.

Table 4-17: Guide for Left-Turn Lane Warrants for Urban and Suburban Arterials

Source: AASHTO A Policy on Geometric Design of Highways and Streets

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Urban and Suburban Arterial Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Urban and Suburban Arterial Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	450	50
10	300	50
15	250	50
20	200	50
25	200	50
30	150	50
35	150	50
40	150	50
45	150	< 50
50 or More	100	< 50
Notes:		
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 4-11.		

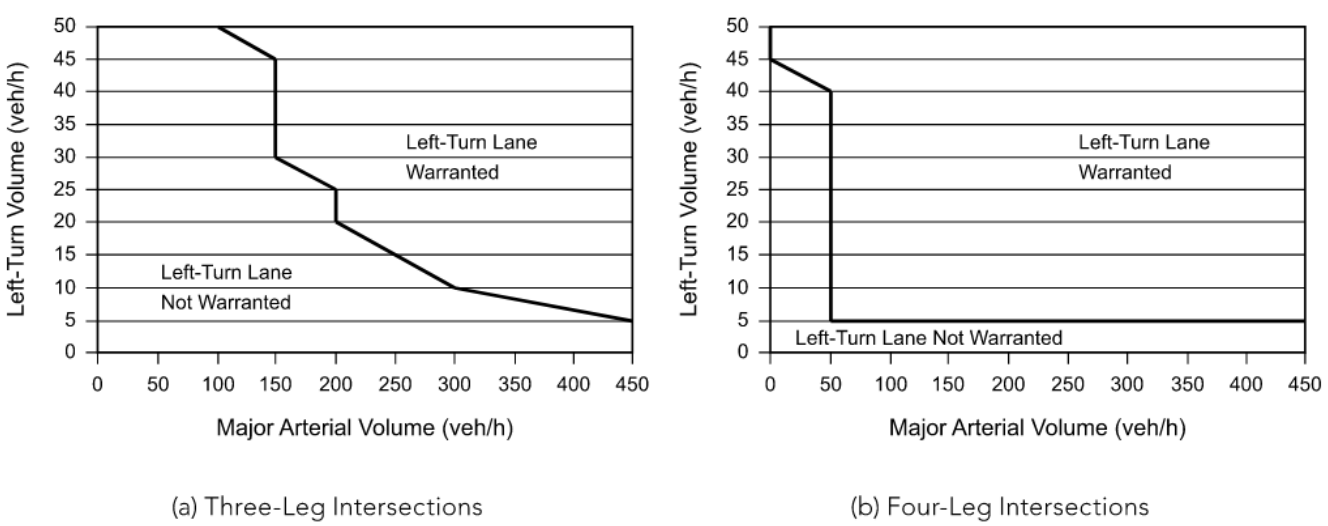
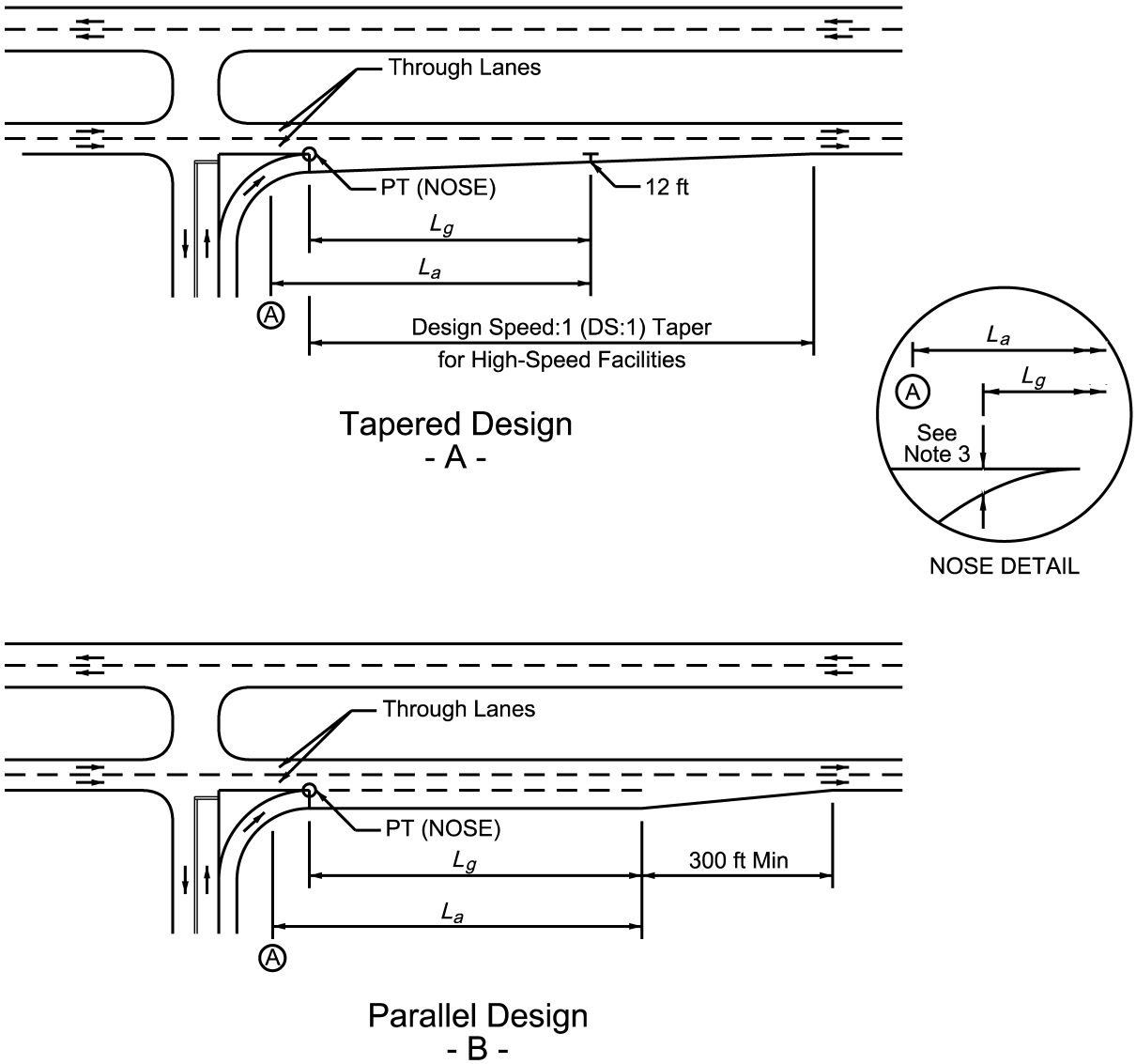


Figure 4-11: Suggested Left-Turn Warrants Based on Results from Benefit-Cost Evaluations for Intersections on Urban and Suburban Arterials

Source: AASHTO A Policy on Geometric Design of Highways and Streets

4.10.2.3 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 4-12**. Recommended acceleration lengths are shown in **Table 4-18**. Adjustments for grade are given in **Table 4-19**.



1. L_a is the recommended acceleration length as shown in Table 4-18 or as adjusted in Table 4-19.
2. Point A is the feature that controls speed on the acceleration lane. L_a 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. L_g 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.

Figure 4-12: Examples of Tapered and Parallel Acceleration Lanes

Table 4-18: Minimum Acceleration Lane Lengths for Entrance Terminals with Flat Grades of Less Than 3 percent

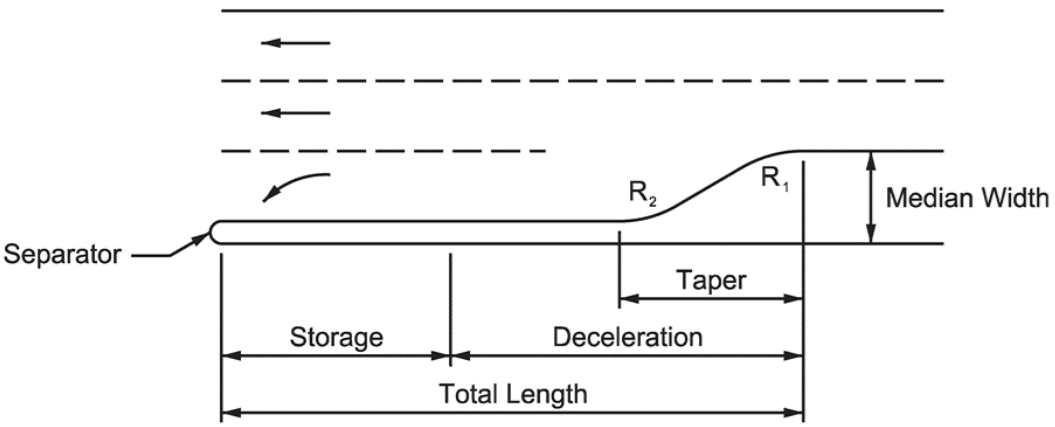
Design Speed of Controlling Feature on Ramp (mph)									
Highway Design Speed (mph)	Stop Condition	15	20	25	30	35	40	45	50
30	180	140	-	-	-	-	-	-	-
35	280	220	160	-	-	-	-	-	-
40	360	300	270	210	120	-	-	-	-
45	560	490	440	380	280	160	-	-	-
50	720	660	610	550	450	350	130	-	-
55	960	900	810	780	670	550	320	150	-
60	1200	1140	1100	1020	910	800	550	420	180
65	1410	1350	1310	1220	1120	1000	770	600	370
70	1620	1560	1520	1420	1350	1230	1000	820	580
75	1790	1730	1630	1580	1510	1420	1160	1040	780
80	2000	1900	1800	1750	1680	1600	1340	1240	980

Table 4-19: Speed Change Lane Adjustment Factors as a Function of a Grade

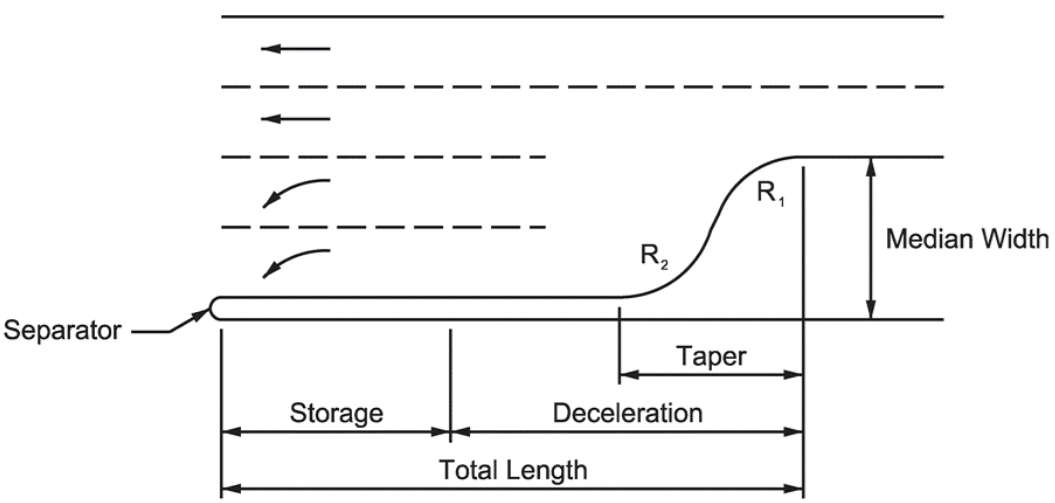
Deceleration Lanes								
Design Speed of Roadway (mph)	Ratio of Length on Grade to Length on Level[1]							
	3 to 4% Upgrade		3 to 4% Downgrade		5 to 6% Upgrade		5 to 6% Downgrade	
All	0.9		1.2		0.8		1.35	
Acceleration Lanes								
Design Speed of Roadway (mph)	Ratio of Length on Grade to Length 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
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

Notes:

1. Ratio in this table multiplied by length of deceleration or acceleration distance in Table 4-18.



SINGLE LEFT-TURN LANE



DUAL LEFT-TURN LANE

Figure 4-13: Left Turn Lane Examples (see Table 4-14)

4.10.2.4 Deceleration Length

Deceleration length, with no speed differential, as shown in **Table 4-14** 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 5 mph and 10 mph speed differential columns in **Table 4-14** for deceleration lengths.

4.10.2.5 Storage Length Calculations

The required storage may be obtained using an acceptable traffic model such as the latest version of the HCM 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, ft

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 per vehicle, ft

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

4.10.2.6 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 4-14** provides recommended lengths for dual left-turn lanes.

4.10.2.7 Right-Turn Deceleration Lanes

Figure 4-14 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 4-14**). 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 4-14**). Refer to the **AMM** for guidelines as to when to consider a right-turn deceleration lane.

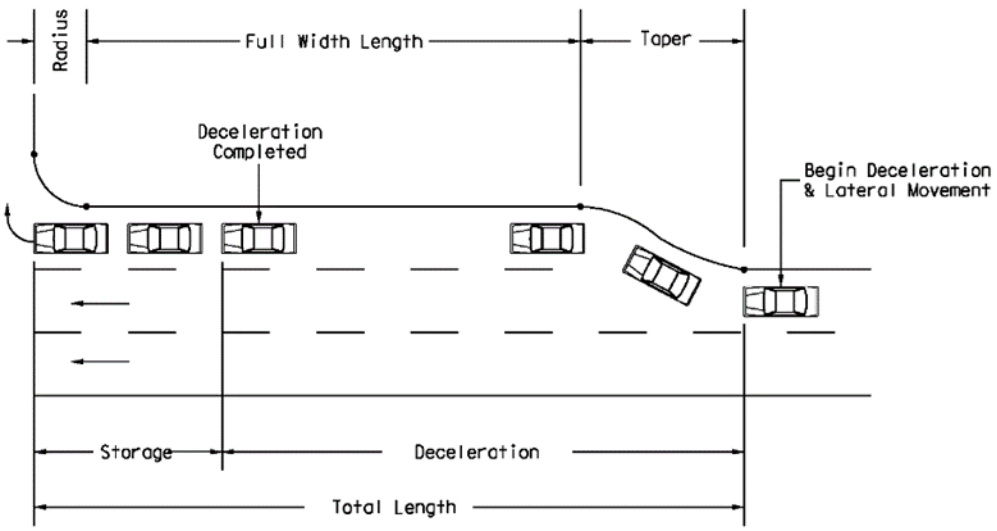


Figure 4-14: Lengths of Right Turn Deceleration Lanes

4.10.2.8 Right Turn Deceleration Lanes on Two-Lane Roadways

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. Refer to the **AMM** for guidelines as to when to consider a right-turn deceleration lane.

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 (i.e., one through lane in each direction of traffic), one median left turn lane, and one right acceleration/deceleration lane. See **Table 4-14** for length of deceleration lanes.

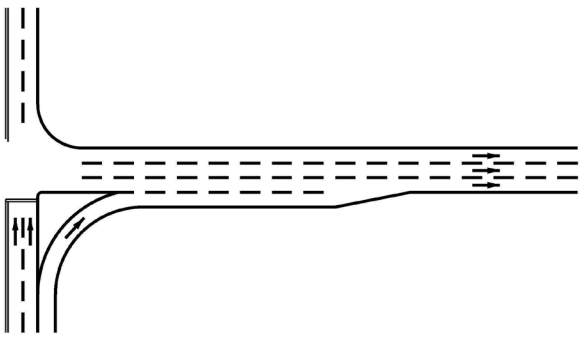
4.10.2.9 Right-Turn and U-Turn Acceleration Lanes

Acceleration lanes are generally not used on urban streets. Right-turn acceleration lanes may be appropriate on some rural highways such as high-volume highways where significant truck percentages are encountered. (see **Figure 4-15 (A)**). **Table 4-18** provides acceleration distances and **Figure 4-12** for taper lengths if an acceleration lane is necessary. Right turn acceleration lanes that end must be yield controlled. This same concept also applies to turnaround lanes (see **Figure 4-15 (B)**).

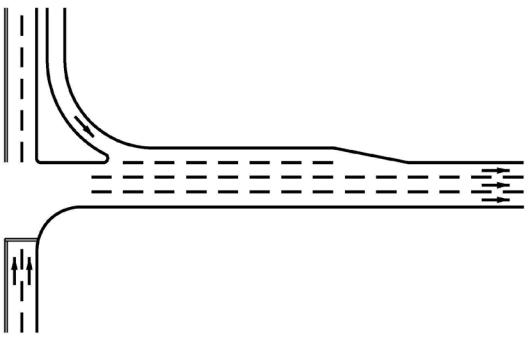
4.10.2.10 Free Right-Turn Lanes

When an acceleration lane does not end, it is considered a free right turn lane. In these cases, stop or yield control may not be provided (see **Figure 4-15 (C)**).

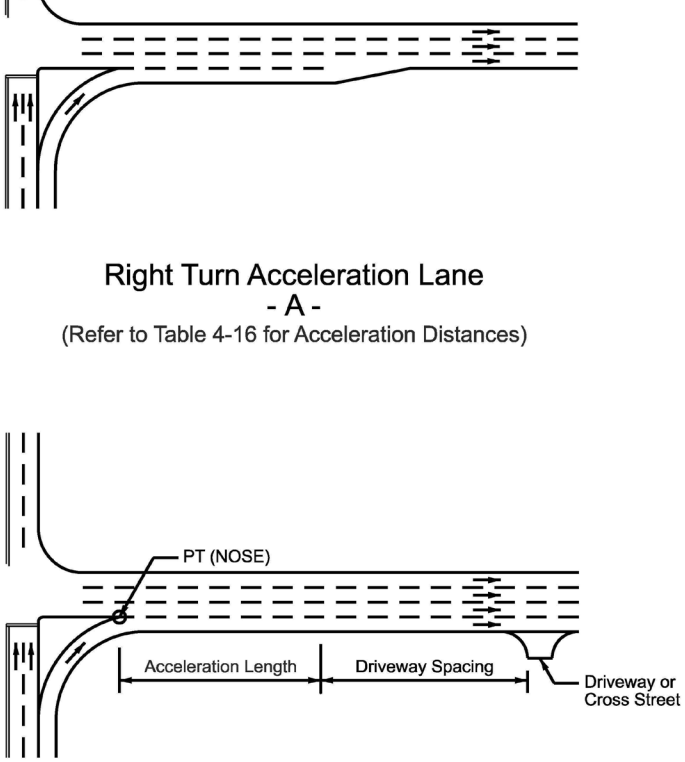
Downstream access should be located so that there is adequate spacing for drivers to accelerate and weave lanes safely to access the connecting driveway or cross street. It is preferred to locate access points far enough downstream so that the acceleration distances shown in **Table 4-18** and **Table 4-19** are met plus the additional driveway spacing shown in the **AMM**. If this **preferred spacing cannot be met, the designer should consider a different type of right turn treatment, or the right turn lane should be yield controlled at the intersection**. A design waiver or design exception is not required if this spacing is not met. Refer to the **AMM** for driveway spacing requirements.



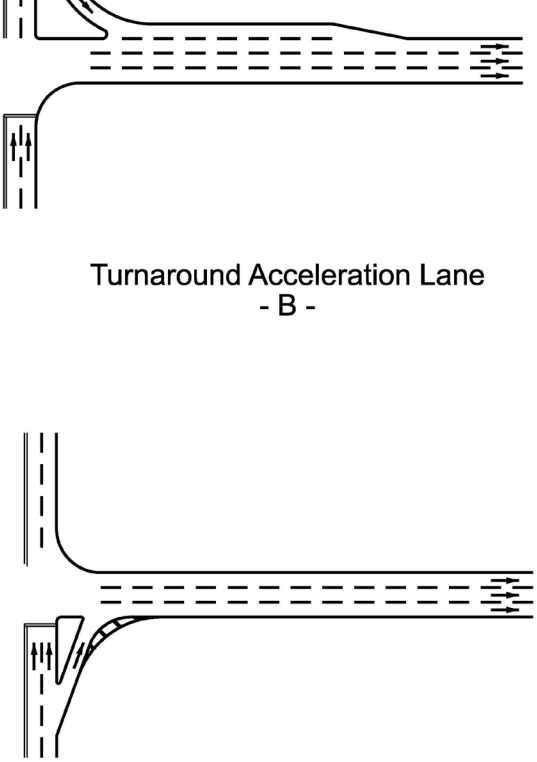
Right Turn Acceleration Lane
- A -
(Refer to Table 4-16 for Acceleration Distances)



Turnaround Acceleration Lane
- B -



Free Right Turn Lane
- C -



Right Turn Slip Lane
- D -

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 Chapter 13 for additional information and guidance on Right Turn Slip Lanes. Figures A and C are not typically designed as right turn slip lanes, however there are several design elements discussed in Chapter 13 that may be incorporated for these types of right turn treatments (e.g. channelizing island, pedestrian considerations, etc.).

Figure 4-15: Types of Right Turn Treaments at Intersections

4.10.2.11 Dual-Right Turn Lanes

High volumes of right-turning vehicles may support a dual right-turn lane configuration to increase capacity for the turns while reducing delay for other movements at the intersection. Dual right-turn lanes can reduce both the length needed for the dual right-turn lanes and the corresponding green time needed for that movement. Dual right-turn movements may be in the form of either two exclusive right-turn lanes or one exclusive right-turn lane and a shared through-right-turn lane (see **Figure 4-16**).

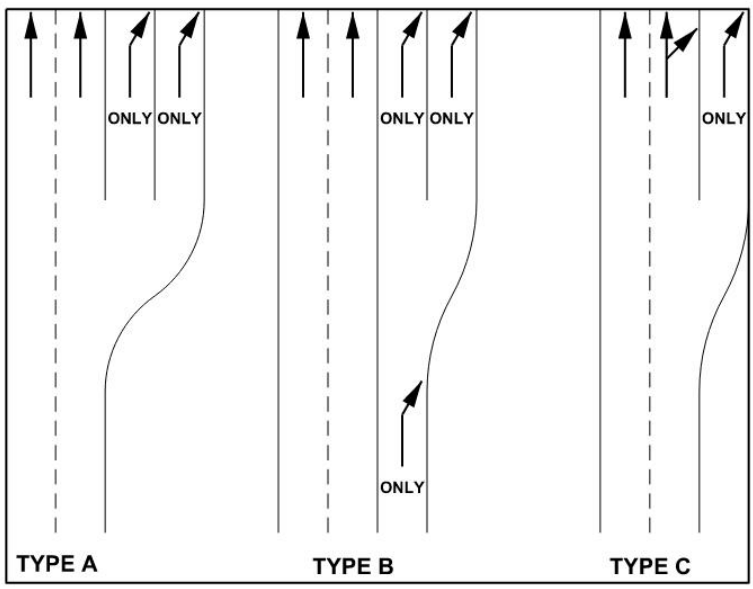


Figure 4-16: Types of Dual Right Turn Lanes

4.10.2.12 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.

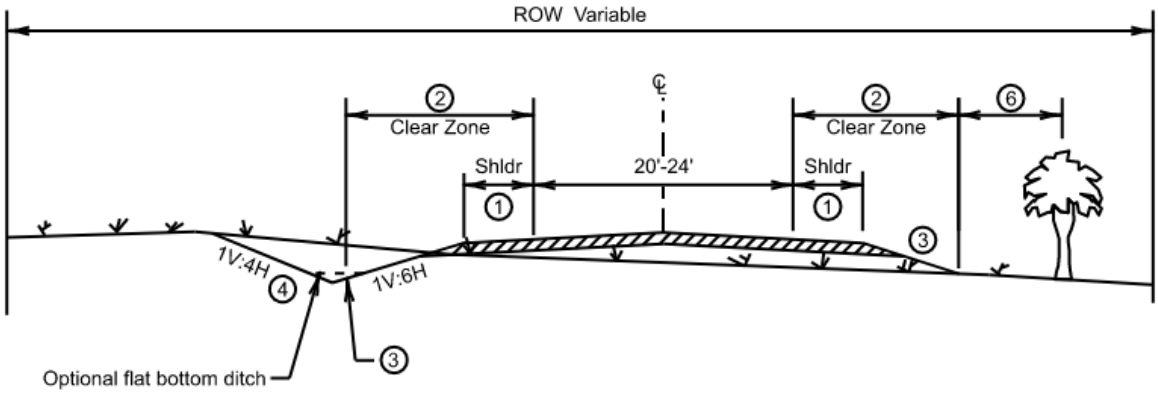
The upgrade flow rate is estimated by multiplying the anticipated or existing design hour volume by the directional distribution factor for the upgrade direction and dividing the result by the peak hour factor (see the **HCM** for definitions of these terms). To calculate the upgrade truck flow rate, 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 preferred taper ratio of 25:1 that should be at least 300-ft long.

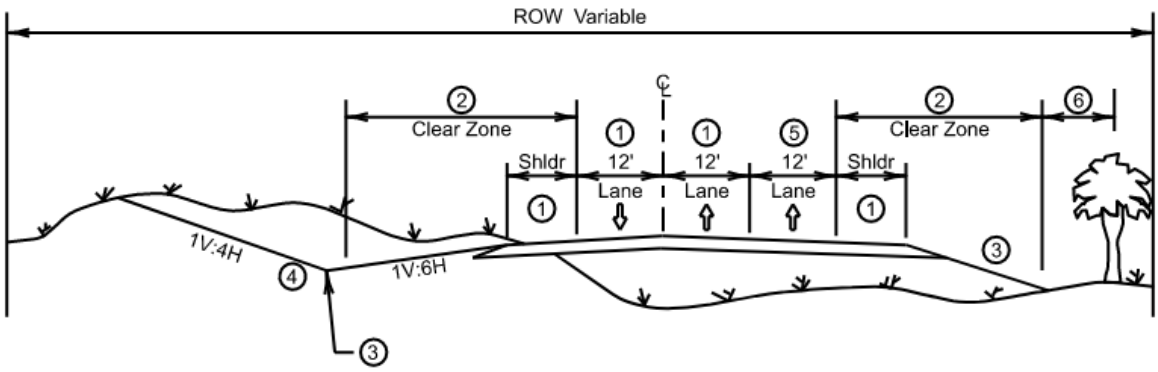
Attention should also be given to the location of the terminal point of the climbing lane. Ideally, the climbing lane should be extended to a point beyond the crest where a typical truck could reach a speed that is within 10 mph of the speed of other vehicles. In addition, climbing lanes should not end 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 **AASHTO's A Policy on Geometric Design of Highways and Streets** for more information regarding the design of climbing lanes.

Figure 4-17 shows a typical section for climbing lanes on rural highways.



TYPICAL SECTION



TYPICAL CLIMBING LANE SECTION

1. See chapter of applicable functional classificaiton for widths of travel lanes and shoulders
2. See Table 4-20 for 4R clear zone widths
3. See 4.10.13 for preferrred ditch sections and 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. Climbing lane width should match through lane width and must not be greater than through lane width
6. Consideration should be given to removing fixed objects 10-ft beyond the toe of slope

Figure 4-17: Climbing Lane Typical Section

4.10.3 Shoulders

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.

Shoulders must be fully surfaced at least for the respective specified minimum width of each functional classification and context classification. Where a greater than minimum width shoulder is provided, partial (not less than minimum width) surfacing or full width surfacing may be provided at the option of the designer. The maximum cross slope of all unsurfaced shoulders must not exceed 1V:10H.

Additionally, projects must be assessed to determine if bicycle accommodations are required on the shoulder per **Chapter 18**, they must meet the additional requirements specified in **Chapter 18**. Bike accessible shoulders must be fully surfaced for the entire width.

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 TxDOT's **Bridge Railing Manual** and the applicable Bridge Railing Standard.

On rural town, urban and urban core collectors 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.

4.10.4 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.

Pavement cross slopes on all roadways should not be less than 1 percent.

In areas of intense rainfall, 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.

Cross-slopes greater than 2 percent should be limited to use in areas of intense rainfall.

In areas of intense rainfall and where three or more lanes are provided in one direction of travel, the maximum cross slope within the tangent section of an alignment should be limited to 4 percent.

Bridge structures with three or more lanes in one direction, should maintain at least 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.

The 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; and
- Turf shoulders should be sloped at about 8 percent.

4.10.4.1 Evaculanes

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. See **Section 21.6** for additional information.

4.10.5 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, see **Table 4-14**.

$$L = WS, \text{ for } S \geq 45 \text{ mph}$$

$$L = WS^2 / 60, \text{ for } S < 45 \text{ mph}$$

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.

4.10.5.1 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.**

4.10.5.2 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.**

4.10.5.3 Lane-Opening Taper (½ L)

Lane-opening tapers are used 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.**

4.10.5.4 Shoulder Taper (1/3 L)

Shoulder tapers are used where the improved shoulders are reduced or increased in width. **The minimum length of a shoulder taper is 1/3 L.**

Note that near the respective project termini, due to site conditions such as tying into to existing grading, drainage constraints, other site-specific circumstances or planned future project tie ins, there may be a reduction in the available length of taper (a design waiver is not required). In such circumstances an emphasis should be placed on the correct placement of pavement markings and signage indicating a reduction or elimination of the shoulder, in accordance with the **Texas Manual on Uniform Traffic Control Devices (TMUTCD)** and applicable Traffic standards.

4.10.5.5 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 4** should be used. **For additional information on temporary traffic control refer to the TMUTCD.**

4.10.6 Rumble Strips

Centerline and shoulder rumble strips are depressed or raised patterns 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 **TxDOT Traffic Standards - RS**. 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 Safer by Design Tool developed by TxDOT's Council on System Safety. Additional information on the **Safer by Design Tool webpage** on TxDOT.gov.

4.10.6.1 Considerations for Centerline and Shoulder Rumble Strip Placement

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 and run-off-road crashes).

Locations where rumble strips must not be placed include:

- Shoulder rumble strips across exit or entrance ramps, acceleration and deceleration lanes, crossovers, gore areas, or intersections with other roadways;
- Depressed rumble strips across bridge decks; and
- Depressed rumble strips on concrete shoulders that will be used as a permanent travel lane or a travel lane in a work zone in the near future.

4.10.6.2 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. See **Chapter 18** for additional guidance on the use of rumble strips with bicycles.

Refer to **TxDOT Traffic Standards – RS** and **FHWA Rumble Strip and Rumble Stripes guidance** for additional information.

4.10.7 Clear Zone

A clear recovery area, or clear zone, should be provided along highways. Clear zone requirements for 4R projects are shown in **Table 4-20**. 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.

The clear zone values shown in Table 4-20 are measured from the edge of travel lane. These are appropriate design values for all cut sections (see **Section 4.12**, 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.

For Freeway mainlanes, or rural arterials with greater than or equal to 750 ADT, the clear zone may be limited to 30 ft. for practical purposes; however, where design speeds are 60 mph and above and side slopes are 1V:5H to 1V:4H, a clear zone of 40 ft is desirable.

Discretionary items which are items that are not necessary for the safety, maintenance, and operation of the roadway (e.g., 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 discretionary items, their foundations and associated appurtenances be placed at least 10-ft beyond the respective minimum clear zone. If this cannot be achieved, if a more desirable alternative location is not available, and if the discretionary item must be placed within the clear zone, it must be built with appropriate breakaway supports or shielded with an appropriate crashworthy device consistent with TxDOT Roadway Standards and applications.

Table 4-20: 4R Clear Zone

Context Classification	Functional Classification	Design Speed (mph)	Avg. Daily Traffic[2]	Clear Zone Width (ft)[1,3,4,5]	
				Minimum	Preferred
Rural	Freeways	All	All	30 (16 ramp)	See 4.10.7
Rural	Arterial	All	≤ 750 > 750	16 30	30 See 4.10.7
Rural	Collector	≥ 50	All	Use above rural arterial criteria.	
Rural	Collector	≤ 45	All	10	16[6]
Rural	Local	All	All	10	16[6]
Suburban	All	All	< 8,000	10[6]	16[6]
Suburban	All	All	8,000 - 12,000	10[6]	20[6]
Suburban	All	All	12,000 - 16,000	10[6]	25[6]
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, Urban Core, Rural Town[7]	All (Curbed)	≤ 45	All	4 from FOC	6 from FOC
Urban	All (Uncurbed)	≥ 50	All	Use above suburban criteria.	
Urban, Urban Core, Rural Town[7]	All (Uncurbed)	≤ 45	All	10[6]	16[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 preferred to provide a 10-ft area free of obstacles beyond the toe of slope.
5. Preferred, rather than minimum, values should be used where feasible.
6. Purchase of 5-ft or less of additional ROW 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 designer may choose to modify clear-zone widths with adjustment factors to account for horizontal curvature, as shown in **Table 4-21**. These modifications are mainly considered when there is a high number of crashes in the area that could be cost-effectively decreased by increasing the clear zone width. Horizontal curves, particularly for high-speed facilities, are usually superelevated to increase safety and provide a more comfortable ride. Increased superelevation on curves where the superelevation is inadequate is an alternate method of increasing roadway safety within a horizontal curve. Snow and ice should be considered in this scenario when increasing the superelevation rate.

$$CZ_C = (L_C)*(K_{CZ})$$

Where:

CZ_C = Clear zone on outside of curvature, ft

L_C = Clear zone width, ft

K_{CZ} = Curve adjustment factor

Table 4-21: Horizontal Curve Adjustment Factor[1]

Radius, ft	Design Speed, mph					
	40	45	50	55	65	70
2,950	1.1	1.1	1.1	1.2	1.2	1.2
2,300	1.1	1.1	1.2	1.2	1.2	1.3
1,970	1.1	1.2	1.2	1.2	1.3	1.4
1,640	1.1	1.2	1.2	1.3	1.3	1.4
1,475	1.2	1.2	1.3	1.3	1.4	1.5
1,315	1.2	1.2	1.3	1.3	1.4	-
1,150	1.2	1.2	1.3	1.4	1.5	-
985	1.2	1.3	1.4	1.5	1.5	-
820	1.3	1.3	1.4	1.5	-	-
660	1.3	1.4	1.5	-	-	-
495	1.4	1.5	-	-	-	-
330	1.5	-	-	-	-	-

Notes:

1. The clear-zone correction factor is applied to the outside of curves only. Corrections are typically made only to curves less than a 2,950-ft radius.

4.10.8 Lateral Offset

Lateral offset is 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.

It is generally preferable that there be uniform clearance between traffic and roadside features such as bridge railings, parapets, retaining walls, and roadside barriers. In an urban environment, ROW 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 over-hang 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.

See **Figure 4-18** for an illustration of lateral offset minimums.

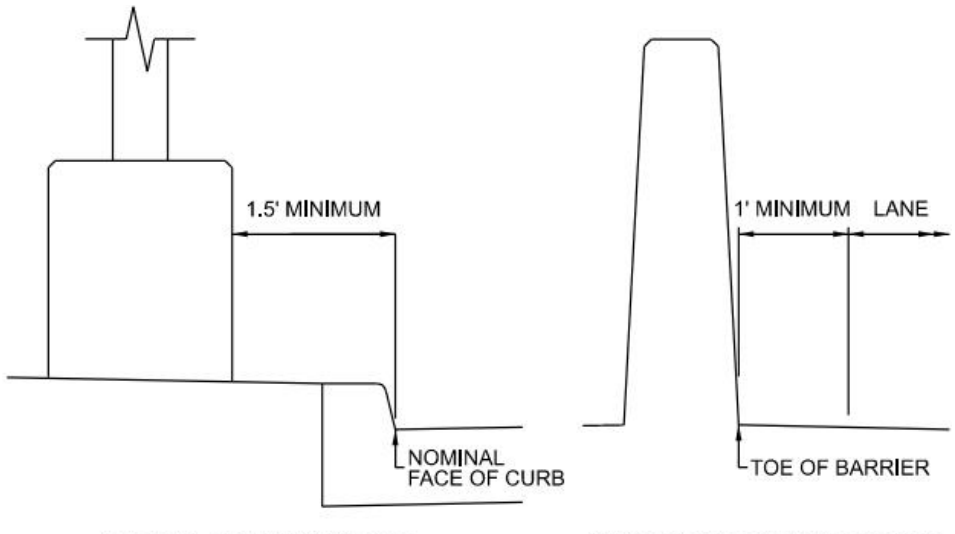


Figure 4-18: Lateral Offset Minimums

A guard fence placed in the vicinity of a curb per the current TxDOT guardrail standard does not violate the minimum lateral offset requirements.

4.10.9 Border and Outer Separation

Border is the area between the roadway and ROW line that may serve several purposes, including but not limited to providing sight distance, a space for separation between pedestrians, bicyclists, and motor vehicle traffic; a sidewalk; or an area for underground and aboveground utilities.

A portion of the border area may be used to accommodate snow storage and may include aesthetic features such as grass or landscaping.

Border widths per classification are detailed below:

- **Local Roads and Streets** a minimum border of 10-ft should be utilized.
- **Collectors**, a border width should be at least 12-ft, including the sidewalk width.
- **Arterials**, a minimum border width should typically be 8-ft, but preferable of 12-ft or more depending on context and nonmotorized user needs.
- **Freeways**, the border should extend beyond the construction limits, where practical, to facilitate maintenance operations, and encourage an effective roadside design.

Every effort should be made to provide wide borders to serve functional needs. It should be carefully considered in the preliminary design stage to ensure that border width issues will not negatively impact PS&E preparation. For example, if 10-ft SUPs are to be installed on the outside of a pair of frontage roads, which are considered collectors, the 12-ft border width would not permit enough space for illumination or power poles, which are set 3-ft inside of the ROW lines. Utilities are just one

important consideration when determining border width.

Target border widths are listed in subsequent chapters per their functional and context classifications.

Outer Separation is the area between the traveled way of through traffic roadway and a frontage road or street.

Such separations function as buffers between the through traffic on the arterial and the local traffic on the frontage road and provide space for a shoulder for the through roadway and ramp connections to or from the through facility. The wider the outer separation, the less influence local traffic will have on through traffic. Wide separations lend themselves to landscape treatment and enhance the appearance of both the highway and adjoining property. A substantial width of outer separation is particularly beneficial at intersections with cross streets because it minimizes vehicle and pedestrian conflicts.

Locations of borders and outer separation along a corridor are detailed in **Figure 4-19**.

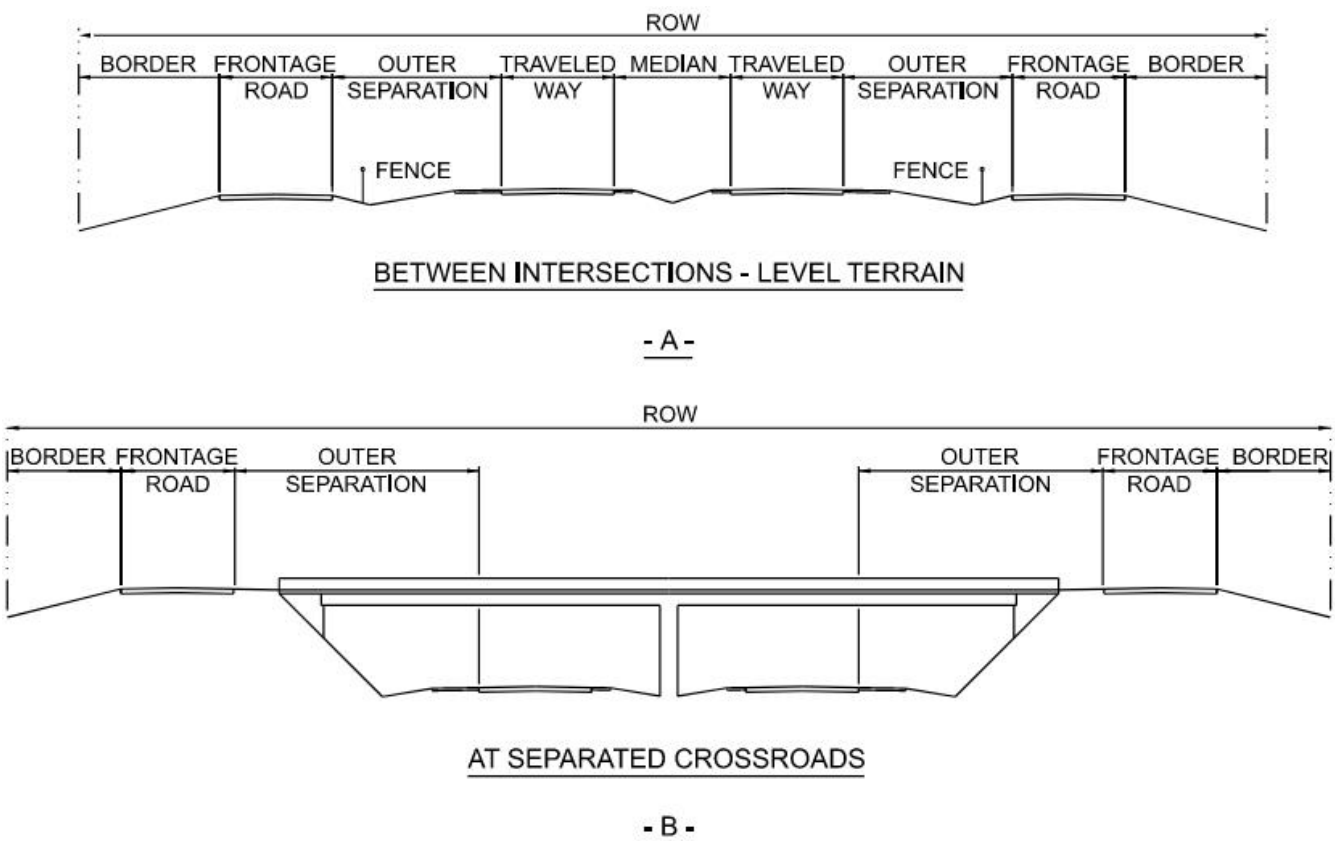


Figure 4-19: Border and Outer Separation Typical Section

Source: AASHTO A Policy on Geometric Design of Highways and Streets

4.10.9.1 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 **Section 4.10.13**.

The first and second type of berms are illustrated in **Figure 4-20**.

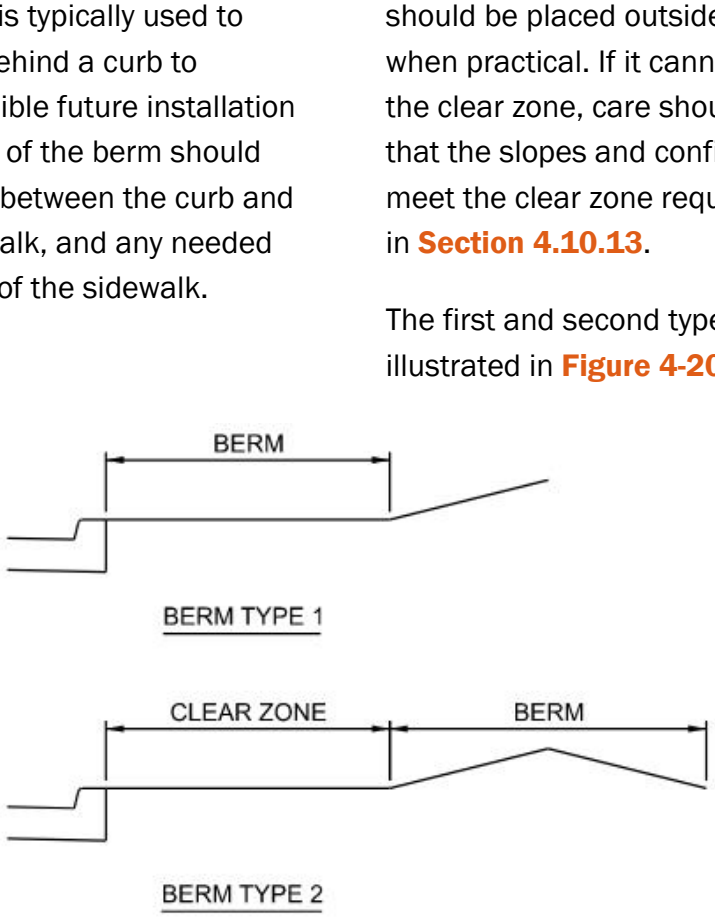


Figure 4-20: Examples of Berms

4.10.10 Right of Way Width

ROW 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 ROW for urban streets is influenced by the following factors:

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

4.10.11 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;
- 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 that includes sight distances greater than minimum, flattened slopes, etc., costs little more over the life of a project and substantially increases safety and operations; and
- 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.

For existing highways, treatment of obstacles should be considered in the following order:

1. **Remove the obstacle.**
2. **Redesign the obstacle so that it can be safely traversed.**
3. **Relocate the obstacle to a point where it is less likely to be struck.**
4. **Make the obstacle breakaway.**
5. **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 safety treating the obstacle is prohibitive.**
6. **Delineate the obstacle (would require a design waiver).**

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.

See **Section 17.3.2** for additional information.

4.10.12 Curbs and Curb/Gutter

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; and
- **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 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.

Curbs are used primarily on frontage roads, crossroads, and low-speed streets in rural town, urban and urban core contexts. **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.

Refer to the current **TxDOT Roadway Standards - CCCG** for illustrations on the various TxDOT standard curb types.

4.10.12.1 Rural

Curbs are not generally used in rural context due to high speeds. If curbs are provided, they must be mountable and should not be closer to

the traveled way than the outer edge of the shoulder.

4.10.12.2 Suburban

Curbs may be used in a suburban context. Mountable curb must be used if the design speed is 50 mph or greater.

4.10.12.3 Rural Town, Urban and Urban Core

Due to limited ROW availability usually associated with these contexts, curbs will generally be used instead of shoulders.

4.10.12.4 Curb Offsets

Curb offsets can vary from 1 to 2-ft. If a curb offset is not provided a design waiver will be required. The curb offset is always measured from the face of curb whether it is a monolithic curb section or a curb and gutter section.

See **Chapter 18** for additional offset requirements for bicycle facilities.

Both variations of curb offsets are illustrated in **Figure 4-21**.

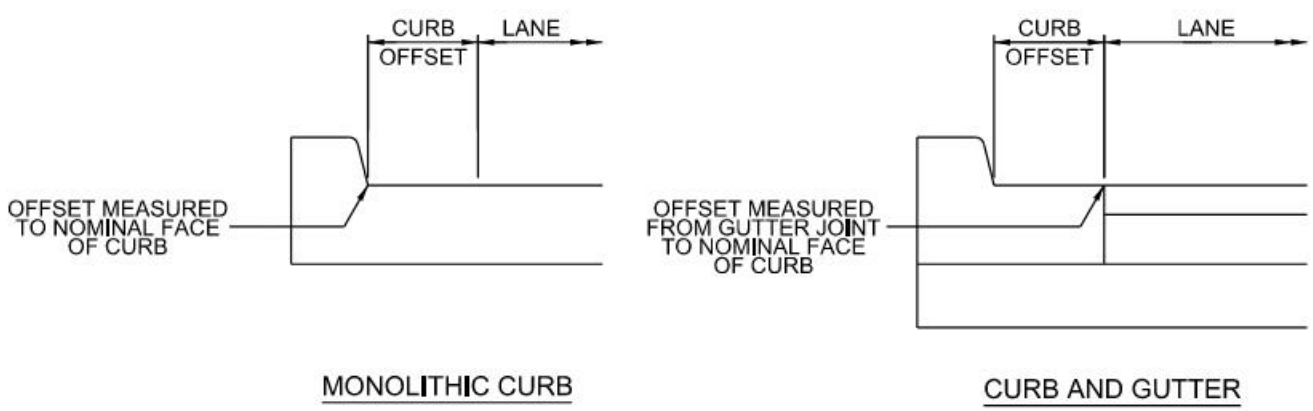


Figure 4-21: Curb Offsets Variations

4.10.13 Side Slopes

Side slopes refer to the slopes of areas adjacent to the shoulder and located between the shoulder and the ROW 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.

A section of typical side slopes is illustrated in **Figure 4-22**.

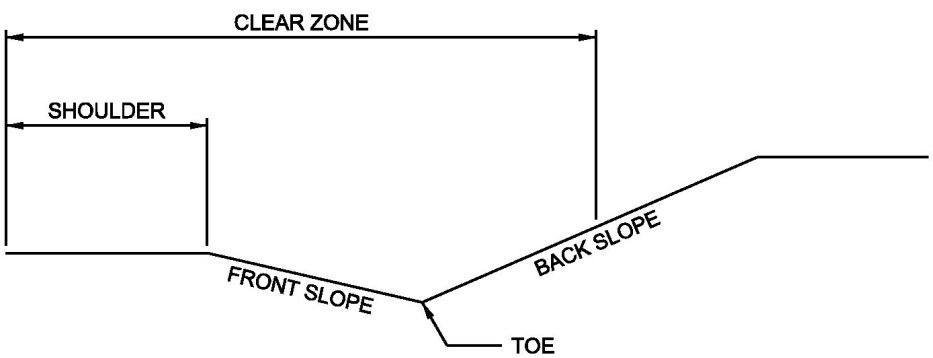


Figure 4-22: Side Slopes Section

4.10.13.1 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 4.10.7.

4.10.13.2 Design Values

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

4.10.13.3 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 riprap. **Slopes greater than 1V:2.5H require evaluation for slope stability.** See TxDOT's **Geotechnical Manual - LFRD** for additional guidance with respect to evaluating slope stability for various conditions.

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 to1V: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 **Chapter 17** for additional information on considerations for barrier need.

4.10.13.4 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**.

4.10.13.5 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, should be at least 6-in below the subgrade crown, and the design year water elevation should not backflow into the pavement structure. For additional information, see Drainage Facility Placement.

For additional information, see **Section 4.12**.

4.10.14 Safety Edge

Straight pavement edge drop-off on roadways have been linked to many serious crashes, including fatal collisions. To mitigate the unsafe pavement drop-off a 1V:1.75H (approximately 30°) safety edge on pavements should be installed when paving or resurfacing roadway. This allows drivers who drift off highways to return to the pavement safely.

In addition to creating a safer driving surface, the safety edge also contributes to an increased pavement durability by reducing edge raveling. During design, the engineer should include the safety edge in typical sections and pavement quantities. See **Figure 4-23** for an example of a safety edge. See the **DES Division (PDS Section) Roadway Design Guidance** webpage for additional guidance.



Figure 4-23: Safety Edge

4.10.15 Medians

A median (i.e., the area between opposing travel lane edges) is provided primarily on divided highways to separate opposing traffic streams. The general range of median width is from 4- to 76-ft. The design width is dependent on the context classification and traffic characteristics 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. See **Chapter 13** for minimum median width required to accommodate various design vehicles.

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.

The types of medians that are commonly used include:

- Depressed;
- Raised;
- Flush; and
- Two-Way Left Turn Lanes (TWLTL).

4.10.15.1 Depressed Medians

Depressed medians are generally found in the rural context and will range from 48- to 76-ft in width. This type of median shares a common ditch between each side of the divided facility and offers an area for storm runoff to collect. This type of median can also provide storage space for snow during snow removal operations.

For multi-lane rural highways without access control, a median width of 76-ft is preferable to provide complete shelter for trucks at median openings. These wide, depressed medians are also effective in reducing headlight glare and providing a clear zone for run-off-the-road vehicle encroachments. They also provide a pleasing appearance and reduce chances of head on collisions. 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. The increased time for vehicles to cross the median can lead to inefficient signal operation.

Areas that are likely to become suburban or urban in nature may require future signalization at intersections. Medians wider than 60-ft should be avoided at these intersections.

Where economically feasible, freeways in rural areas should also desirably include a 76-ft depressed 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 ROW costs are prohibitive, reduced median widths (less than 76-ft) may be used. 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.

4.10.15.2 Raised Medians

A raised median is used 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.

This type of median can typically be found on roadways in the suburban, urban, or urban core contexts and may range from 4 to 36-ft in width. **On high-speed facilities, mountable curb should be used for the raised median.**

A raised median design should be considered where:

- ADT exceeds 20,000 VPD;
- New development is occurring, and volumes are anticipated to exceed 20,000 VPD; 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 width of 16-ft will effectively accommodate left-turning traffic and a 4-ft median divider measured from the face of curb. 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). See **Chapter 19** for additional guidance.

An 18-ft median is preferred to accommodate the 6-ft pedestrian refuge in the median, rather than providing 10-ft left turn lanes.

An illustration of this difference is shown in **Figure 4-24**.

To prevent recurring damage to the raised median divider, the divider should be at least 2-ft wide (measured from face of curb 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.

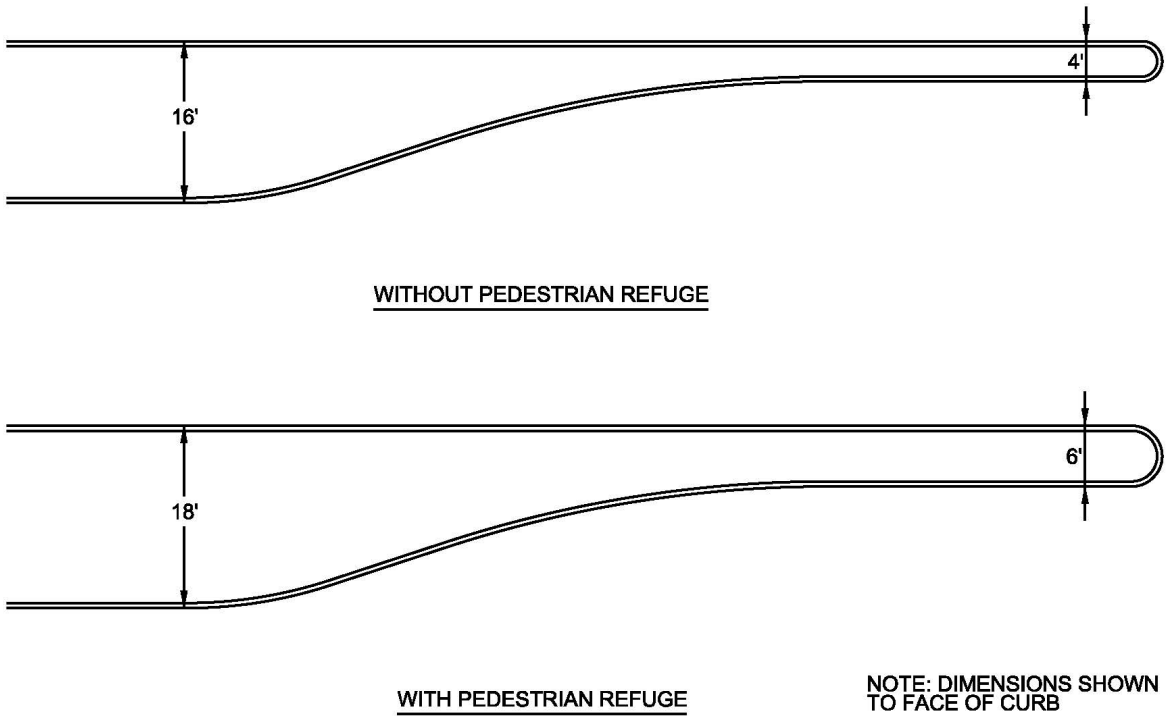


Figure 4-24: Median Left Turn Lanes with and without Pedestrian Refuge

4.10.15.3 Flush Medians

Flush medians are medians that can be traversed. A flush median cannot prevent left-turn and cross maneuvers since it can be easily crossed and cannot physically prevent these types of movement. Therefore, for roadways where access control is desirable, flush medians should not be used. 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.

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.

This type of flush median is generally located on roadways in the rural or suburban contexts with roadside development and may range from 4- to 16-ft in width. 4-ft wide flush medians provide little separation of opposing traffic and minimal refuge area for pedestrians. 14- to 16-ft wide flush medians offer space for traffic turning left but does not offer protection for crossing vehicles.

Urban and suburban freeways generally include narrower, flush medians with continuous longitudinal barriers. The median width may vary from 24- to 30-ft, with 24-ft commonly used. See **Chapter 17** for recommendations on the use of median barriers. See **Chapter 8** for additional information on freeway medians.

4.10.15.4 Two-Way Left-Turn Lanes

TWLTL are flush medians that may be used for left turns by traffic from either direction on the roadway. The TWLTL is appropriate where there are operational concerns for a high frequency of mid-block turns exist or are anticipated, 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 4-22**. When applying this criterion to new location projects or on reconstruction projects the corresponding preferable width should be used where widening necessitates the removal of exterior curbs. Minimum values shown in **Table 4-22** are appropriate for restrictive ROW 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 4-22: Median Lane Widths for Two-Way Left-Turn Lanes

Design Speed (mph)	Width of TWLTL (ft)	
	Preferable[1]	Minimum
≤ 40	14	11
45 or 50	14	12
>50	16	14
Notes:		
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 the facility meets the following criteria:

- Future ADT volume of greater than:
  - 3,000 VPD for an existing two-lane facility,
  - 6,000 VPD for an existing four-lane facility, or
  - 10,000 VPD for an existing six-lane facility; and
- Side roads plus driveway density of 20 or more entrances per mile for both directions of travel for urban context or 10 or more entrances per mile for both directions of travel for suburban context.

All cross sections should be evaluated for pedestrian crossing capabilities. See **Chapter 19** for additional guidance.

4.10.15.5 Median Openings

For rural town, suburban, urban, and urban core contexts, 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.

As the number of median openings 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.

Directional openings, pictured in **Figure 4-25**, can be used to limit the number and type of conflicts.

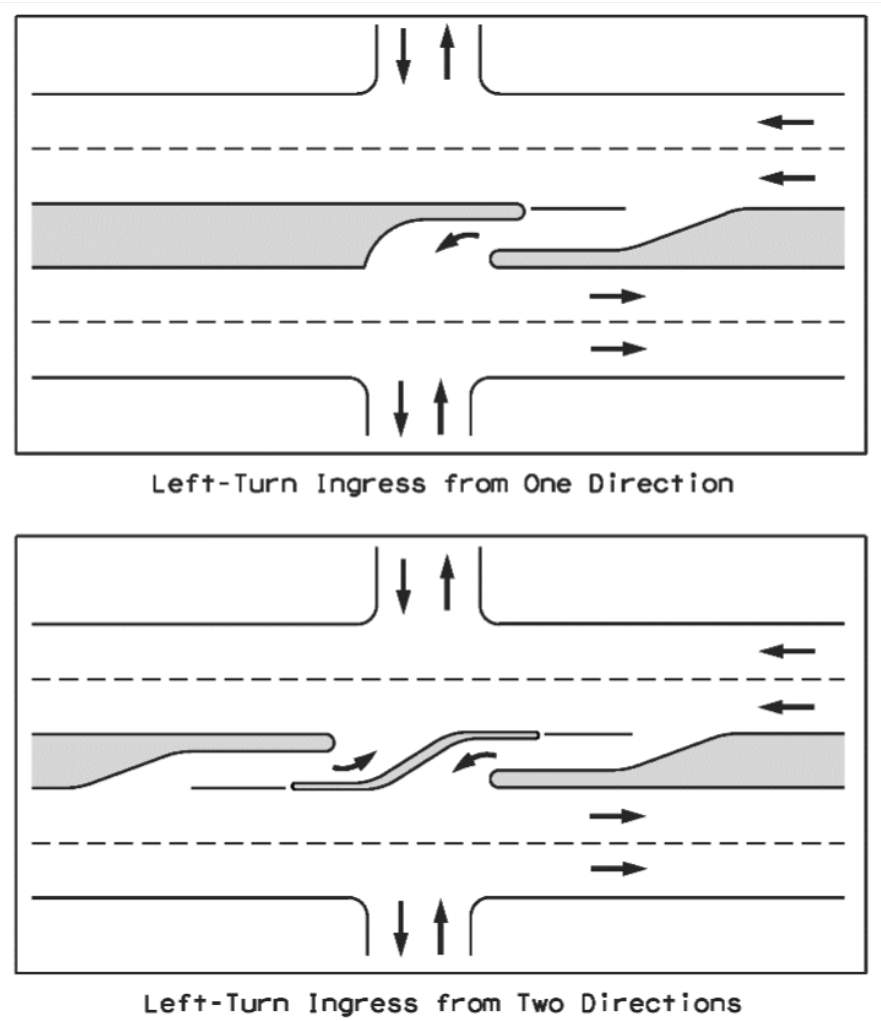


Figure 4-25: Types of Directional Openings

The positive offset design shown in **Figure 4-26(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 A Policy on Geometric Design of Highways and Streets**.

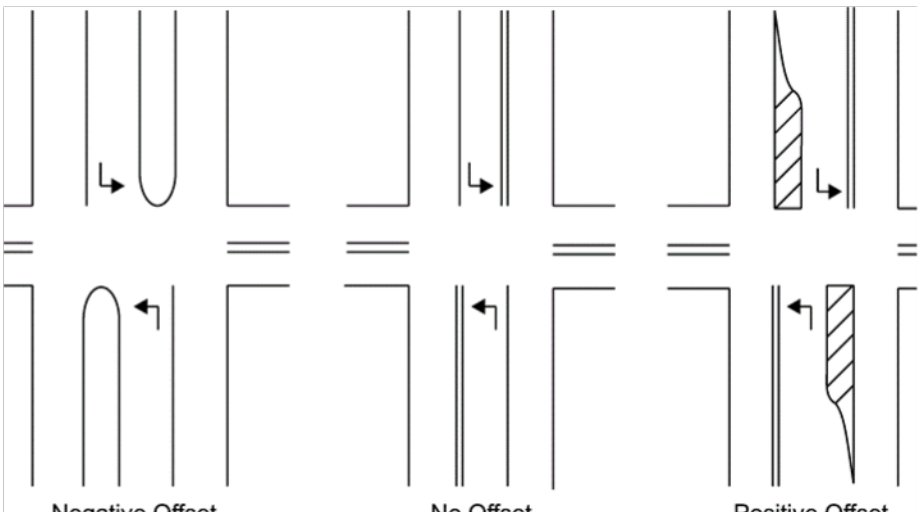


Figure 4-26: Examples of Left-Turn Lanes with Negative, Zero, and Positive Offset

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.

See **Figure 4-27** for an illustration of these two types of median ends.

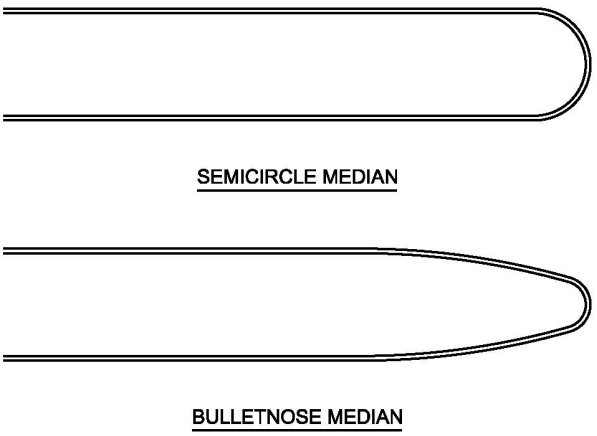


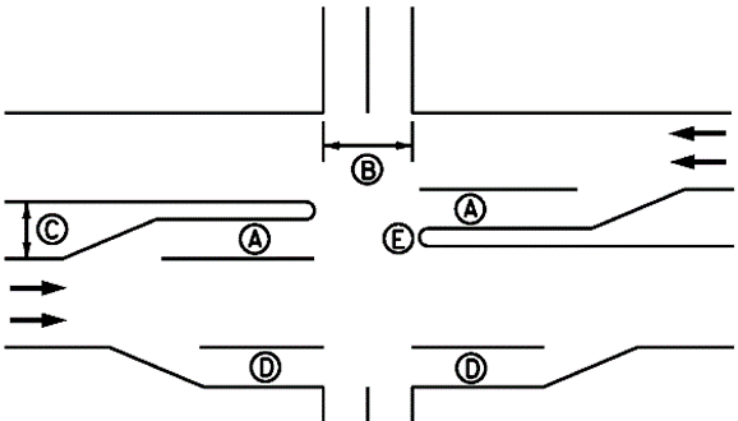
Figure 4-27: Types of Median Ends

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.

On rural divided highways, closely spaced median openings 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 4-28**, rural divided highways should provide left-turn lanes 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 TxDOT's **AMM** "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 mainlane alignment, particular care should be exercised at median openings to provide a satisfactory profile along the crossover with flat approaches to the mainlanes.



Ⓐ Median turn lane

Ⓑ Median opening (varies based on number of cross road lanes and design vehicles)

Ⓒ Median

Ⓓ Right speed change lane (acceleration or deceleration)

Ⓔ Median nose

Figure 4-28: Multi-Lane Rural Highway Intersection

Median openings should be at least 40-ft wide, or the width of the crossroad pavement plus 8-ft. **Design vehicles are often used as the basis for minimum design of median openings, particularly for multilane crossroads and skewed intersections.** See **Table 4-1** for additional information.

4.10.15.6 Median Encroachment Design Guidance and Countermeasures

Median encroachment countermeasures should be considered where appropriate. High severity injuries and fatalities are a result of cross median crashes on high-speed 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.

4.10.16 Parking

This section discusses the features and design criteria for parking facilities and includes the following subsections:

- Fringe parking lots; and
- Parking along highways and arterial streets.

4.10.16.1 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.

4.10.16.2 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 ROW 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.

4.10.16.3 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:
    a. Disabled persons;
    b. Bicycles;
    c. Motorcycles;
    d. Vehicles of park and ride users; and
    e. 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.

Authority and Funding

For fringe parking areas within highway ROW, projects are generally developed as any other multiple use project. When parking lots are proposed outside of existing or proposed highway ROW, commission approval is required.

Park and pool lots are eligible for Federal-aid participation. Projects are usually located within or adjacent to highway ROW 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.

4.10.16.4 Parking Along Freeways and Arterial and Collector Streets

This section deals with parking as it pertains to the frontage roads of a controlled access highway and parking along urban and suburban arterials and collectors. Off-street parking

facilities provided within highway ROW are discussed in the previous **Section 14.10.16.1**. Rest areas as parking facilities are not considered in this section.

4.10.16.4.1 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.

4.10.16.4.2 Curb Parking

In general, curb parking on arterial and collector 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, collectors 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.;
- 12- to 14-ft parking lane widths should be provided if bicyclists, as well as parked vehicles, need to be accommodated; 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.

4.10.16.5 Parking Along Local Roads

Parking along local roads in the rural and suburban contexts is not permitted.

Curb parking lane widths in rural town, urban and urban core contexts should be 8- to 9-ft. In non-commercial and non-industrial areas, a 7-ft minimum width may be used.

4.10.17 Retaining Structures

Retaining structures are used where vertical drops create lateral earth pressure. Examples of these structures include gravity walls, cantilevered walls, and mechanically stabilized earth walls.

Retaining structures contain and support soil where loadings may otherwise cause the soil to move and impact the structure supported by it. Depending on the location and specifications, these structures may be concrete or altered soil. Refer to TxDOT's **Geotechnical Manual – LRFD** for additional information on geotechnical design for retaining structures.

Selection of retaining structures will depend on factors such as ROW limitations, foundation

soil characteristics, loading requirements, economic feasibility, and aesthetics. Whether the section is in cut or fill should also be considered, as some retaining structures are better suited for one or the other.

In situations where a roadway is elevated along a retaining wall, the drop off condition must be protected for the entire length of the wall, regardless of if the retaining wall is inside or outside the clear zone. Refer to TxDOT's **Bridge Railing Manual** for additional information on protection requirements for a roadway elevated along a retaining wall.

A more detailed discussion on design calculations and geometric specifics can be found in the **AASHTO LRFD Bridge Design Specifications**.

4.10.18 Utilities and Easements

Where utility work extends beyond the ROW boundaries, easements may be used to accommodate those activities. These easements may either be perpetual or temporary, depending on the work being done and expected maintenance once the work is completed.

Title reports and easement documents should be obtained and reviewed to determine boundaries and specifics of property rights of parcels and utilities affected by proposed work.

Compensation for utility adjustments may be affected not only by easements, but also by the ratio of the utility which exists within or outside of the easement and existing ROW.

For more information regarding utilities in relation to ROW and easements, refer to TxDOT's **ROW Utilities Manual**.

4.11 Sight Distance

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.

4.11.1 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 percent 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 a passenger car.

The calculated and design stopping sight distance are shown in **Table 4-23**.

The values given in **Table 4-23** 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.

The calculated corrections for these types of factors are given in **Table 4-24**. The computed distances for various speeds at the assumed conditions on level roadways are developed from the equations below.

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

Where:

SSD = stopping sight distance, ft

V = design speed, mph

T = brake reaction time, 2.5 s

A = deceleration rate, ft/s^2

When a highway is on a grade the equation for braking distance is modified to include the grade as shown below:

$$d_B = \frac{V^2}{30\left[\left(\frac{a}{32.2}\right) \pm G\right]}$$

Where:

d_B = braking distance on grade, ft

V = design speed, mph

A = deceleration, ft/s^2

G = grade, rise/run, ft/ft

The stopping sight distances for various grades are shown in **Table 4-24** these values are determined using the equation above.

Table 4-23: Stopping Sight Distance on Level Grade

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

Notes:

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

Table 4-24: Stopping Sight Distance on Grade

Design Speed (mph)	Stopping Sight Distance (ft)					
	Downgrades			Upgrades		
	3%	6%	9%	3%	6%	9%
15	80	82	85	75	74	73
20	116	120	126	109	107	104
25	158	165	173	147	143	140
30	205	215	227	200	184	179
35	257	271	287	237	229	222
40	315	333	354	289	278	269
45	378	400	427	344	331	320
50	446	474	507	405	388	375
55	520	553	593	469	450	433
60	598	638	686	538	515	495
65	682	728	785	612	584	561
70	771	825	891	690	658	631
75	866	927	1003	772	736	704
80	965	1035	1121	859	817	782

4.11.2 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 4-25** shows recommended decision sight distance values for various avoidance maneuvers.

Table 4-25: Decision Sight Distance

Design Speed (mph)	Avoidance Maneuver Decision Sight Distance (ft)				
	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 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; and
- 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.

4.11.3 Passing Sight Distance

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/s^2, 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 returns 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 reduce the safety and LOS 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 ROW line.

Minimum passing sight distance values for design of two-lane highways are shown in **Table 4-26**. **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 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 4-26: Passing Sight Distance

K-Values for Determining Length of Crest Vertical Curve for Various Passing Sight Distances (PSD)		
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
Notes:		
1. Based on eye height = 3.5 ft and object height = 3.5 ft; K = PSD2 / 2800.		

4.11.4 Intersection Sight Distance

Intersection sight distance is fundamental to intersection operation and **should be calculated at every intersection** to ensure a safe and traversable intersection. Calculations and more information about intersection sight distance can be found in **Chapter 13**.

4.11.5 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.

When the minimum sag vertical curve length based on headlight sight distance, as described in **Figure 4-29**, is achieved for vertical clearances of 14-ft or higher, then **Figure 4-29** and subsequent equations are not applicable. However, if any of the following conditions occur, then the under-crossing curve length as shown in **Figure 4-29** and subsequent equations should be verified based on the applicable stopping 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; and
- The vertical clearance is less than 14-ft.

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)

$$L = \frac{AS^2}{800\left[C - \frac{h1+h2}{2}\right]}$$

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):

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

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

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

Where:

L = length of vertical curve, ft

S = sight distance, ft

C = vertical clearance, ft

h_1 = height of eye, ft

h_2 = height of object, ft

A = algebraic difference in grades, percent

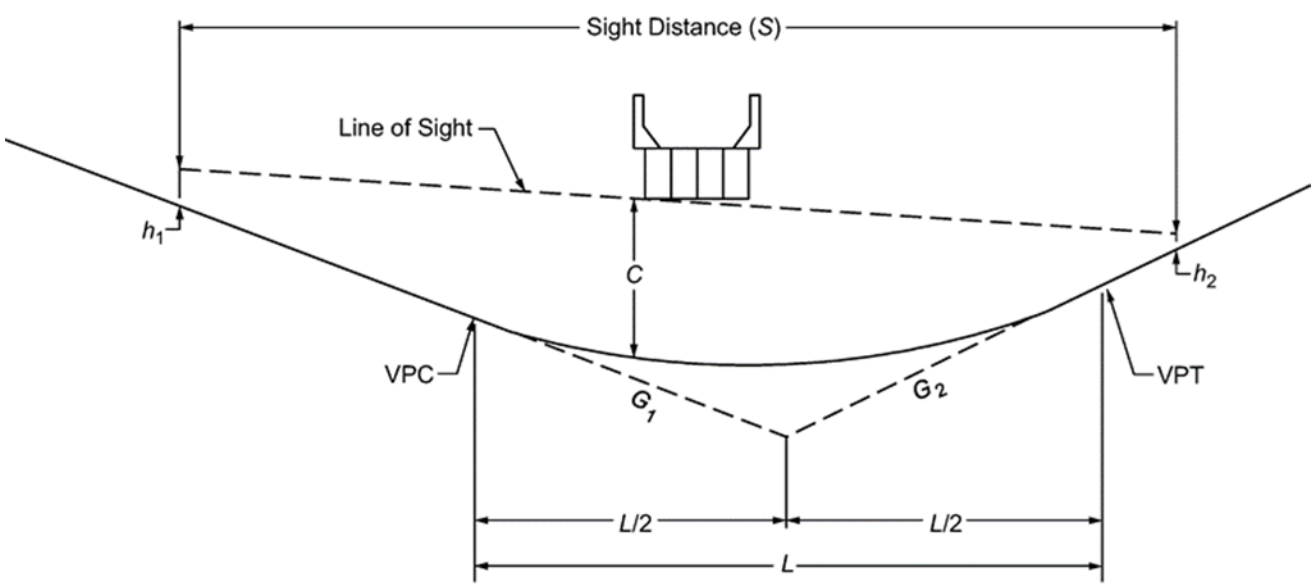


Figure 4-29: Sight Distance at Under-Crossings

Source: AASHTO A Policy on Geometric Design of Highways and Streets

4.12 Drainage Facilities

In designing drainage systems, the primary objective is to properly accommodate surface runoff as well as cross drainage through highway ROW with 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 collisions; 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 4-20** and the discussions regarding Side Ditches, Roadside Design, and Clear Zone. Preferable 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, hazardous 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.

4.12.1 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, ROW availability, and other hydraulic 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 4.10.13**. Where ROW availability and economic conditions permit, flatter slopes may be used.

Design values for 4R clear zones are shown in **Table 4-20** for new location and 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.

4.12.2 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 4-27**.

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:4H or flatter within the clear zone, 1V:3H or flatter outside the clear zone, 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.

Table 4-27: Maximum Diameter of Small Pipe Culvert

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

4.12.3 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 in and a total opening (i.e., distance) of 20-ft or less between extreme inside faces as measured along the highway centerline. 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:4H or flatter within the clear zone, 1V:3H outside the clear zone, 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 4-20**. Where an improved design is warranted using **Table 4-20**, the removal of headwalls and installation of sloped ends with safety pipe runners is the preferred safety treatment.

In certain situations, treatment with safety pipe runners may be impractical (e.g., culvert skew exceeds 15 degrees or severe debris problems). For these conditions, locating intermediate size culvert ends to meet desirable clear zone values (see **Table 4-20**) 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. The orientation of pipe runners must also consider the direction of mainlane traffic and be selected properly for cross drainage or parallel drainage.

4.12.4 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 ROW 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.

4.12.5 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 of the clear zone. Exceptions to this requirement can be obtained by approval of a Design Exception. See TxDOT's **Bridge Railing Manual** for additional information on protection of bridge class culverts.

See **Figure 4-30** for a visual representation of bridge class culverts.

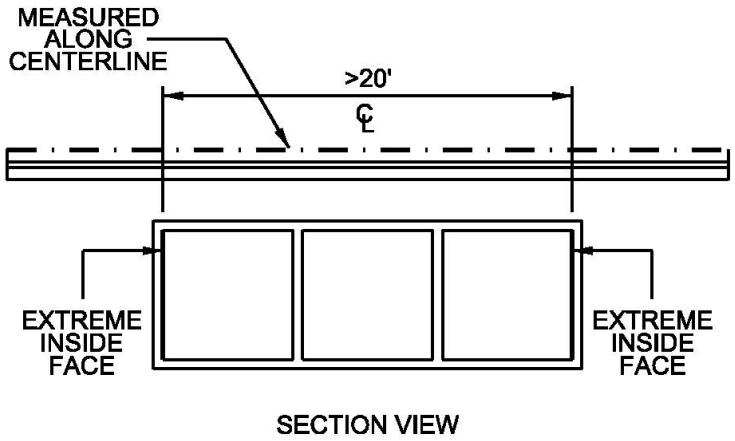


Figure 4-30: Bridge Class Drainage Culvert

Recommended treatment options are in the following priority:

1. Safety treat culvert ends.
2. Bridge Railing is an option for all depths of cover, **Table 4-28** provides available TL-3 guardrail options consistent with the respective guardrail and attenuator standard.

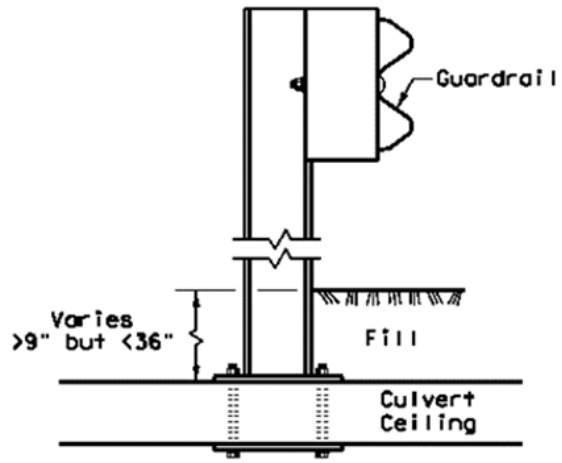
See discussion on cost comparison under Multiple Box Culverts for considerations when deciding on whether to safety treat or shield the culvert ends.

Table 4-28: Treatment Barrier for Bridge Class Culverts

Depth of Cover (D)	Treatment
D < 9-in[1]	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
Notes:	
1. Refer to TxDOT Bridge Railing Manual 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 4-31** and the current guardrail standard for specific application.



Note: The use of this post is limited to culverts with fill greater than 9" but less than 36".

LOW FILL CULVERT POST

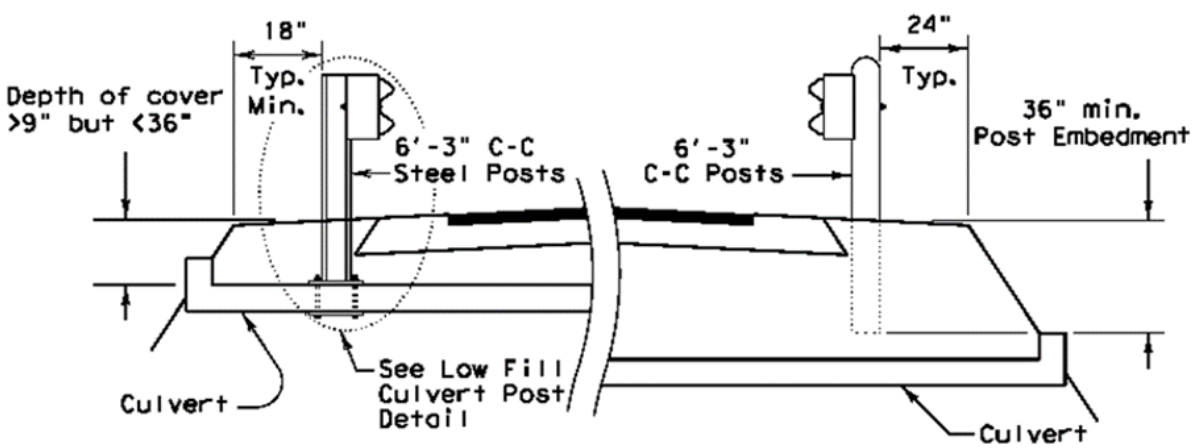


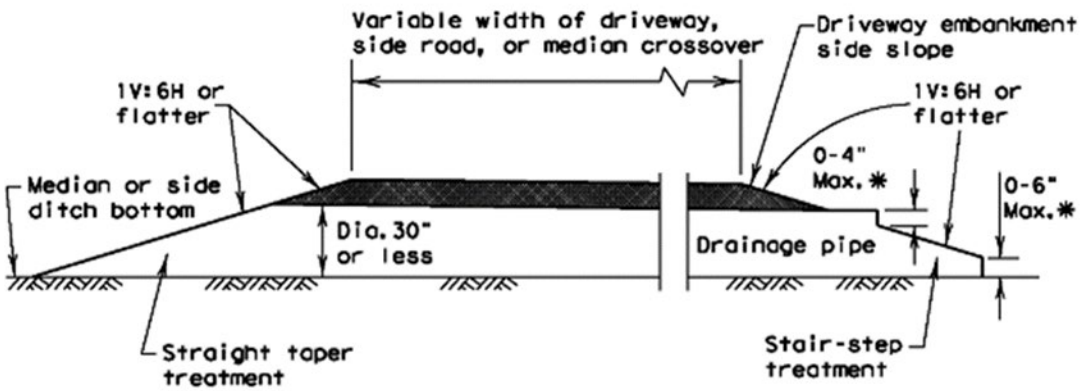
Figure 4-31: Use of Guardrail at Culverts

4.12.6 Parallel Drainage Culverts

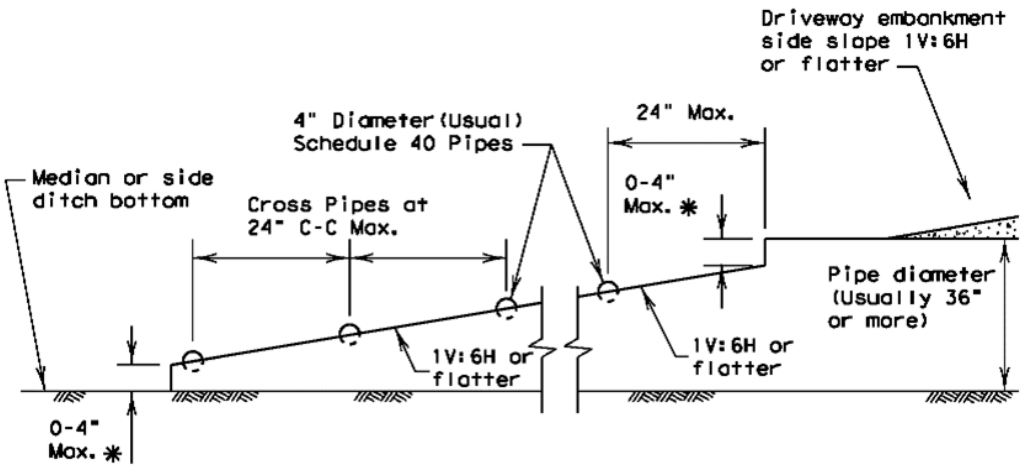
The inlet and outlet points of culverts handling drainage parallel to the travel lanes, such as at driveways, side roads, and median crossovers, are concerns in providing a safe roadside environment. Flow quantities for parallel drainage situations are generally low with drainage typically accommodated 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 4-32**.
- 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. Typical details for a driveway, side road, or median crossover grate are shown in **Figure 4-32**. 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 if adequate conveyance can be provided.
- 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.



USE OF SLOPING PIPE ENDS WITHOUT CROSS PIPES



USE OF SLOPING PIPE ENDS WITH CROSS PIPES

Figure 4-32: Sloping Pipe Ends

4.12.7 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 4-29** are preferred and should especially be sought when the ditch location is within the clear zone requirements. Where conditions dictate, such as insufficient existing ROW 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 **Chapter 17**.

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.

Table 4-29: Preferable Ditch Slopes

Given Front Slope (Vertical:Horizontal)	Preferable 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.		

4.13 Environment

A roadway has wide-ranging effects in addition to providing traffic service to users. It is essential that the highway be considered as an element of the total environment. **The roadway can and should be located and designed to complement its environment and serve as a catalyst to environmental improvement.**

The area surrounding a proposed road or street is an interrelated system of natural synthetic, and sociologic variables. Changes in one variable within this system cannot be made without some effect on other variables. Because roadway location and design decisions affect the development of adjacent areas, it is important the environmental variables be given full consideration. Also, care should be exercised so that applicable local, State, and Federal environmental requirements are met.

It is essential that environmental considerations and constraints are identified early in the process through the development of the environmental work plan. The work plan will outline the necessary investigations to determine potential impacts to assist in avoidance, minimization, or mitigation of those impacts.

4.14 Human Factors

4.14.1 Older Drivers & Pedestrians

Older drivers and pedestrians are a significant and growing segment of the road user population and should be accommodated in the design of road and pedestrian 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 most practical enhancement to be considered when related to better accommodate older drivers is to increase the sight distance of the facility, which may be accomplished through increased use of decision sight distance. The gradual aging of the driver population suggests that increased use of decision sight distance may help to reduce future crash frequencies for older drivers.

The factors that could improve roadway and traffic control features for making driving easier for older drivers include:

- Design for the 95th or 99th percentile driver where appropriate;
- Improve sight distance by modifying designs and removing obstructions;
- Provide decision sight distance in advance of key decision points;
- Simplify intersections and interchanges that involve multiple information reception and processing;
- Consider alternate designs to reduce conflicts;
- Increase use of protected left turns;
- Increase vehicular clearance times at signalized intersections;
- Increase walk times for pedestrians;
- Provide wider and brighter pavement markings and paint median noses;
- Provide larger and brighter signs;
- Reduce sign clutter;
- Provide information such as advance warning signs for upcoming traffic movements;
- Provide rumble strips or profile pavement markings;
- Provide intersection channelization; and
- Reduce intersection skew.

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.

4.15 Roadway 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 TPP. 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 4.7**, **4.8**, and **4.10** 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 5
Local Roads (4R)

5.1 Design Considerations

5.1.1 Introduction

Local roads primarily serve to provide access to abutting land. Local roads provide high access and limited mobility. Design criteria for local roads is of lower order due to the relatively low traffic volumes and high proportion of highway mileage. However, they should be planned, located, and designed to be suitable for predictable traffic operations and should be consistent with the development and environment abutting the ROW.

This chapter discusses the features and design criteria for local roads and includes the following topics:

5.1 Design Considerations

- 5.1.1 Introduction;
- 5.1.2 Target Design Values;
- 5.1.3 Design Speed;
- 5.1.4 Design Traffic Volumes;
- 5.1.5 Levels of Service;
- 5.1.6 Travel Lane Width and Number of Lanes;
- 5.1.7 Shoulders;
- 5.1.8 Speed ane Changes;
- 5.1.9 Parking Lanes;
- 5.1.10 Intersections;
- 5.1.11 Medians;
- 5.1.12 Curbs;
- 5.1.13 Cross Slope and Superelevation;
- 5.1.14 Grades;
- 5.1.15 Roadside Design;
- 5.1.16 Structures;
- 5.1.17 Vertical Clearance; and
- 5.1.18 Bicycle and Pedestrian Facilities.

5.2 Rural Design Element Characteristics

5.3 Rural Town, Suburban, Urban and Urban Core Design Element Characteristics

This chapter is organized with an introductory section on the general design considerations for local roads, followed by separate design discussions for local roads in rural, rural town, suburban, urban, and urban core contexts.

5.1.2 Target Design Values

Table 5-1 shows target design values for Local Roads in each context classification. These design values may be adjusted based on the anticipated operating speed and a performance-based practical design evaluation of the corridor as discussed in **Chapter 2**.

The target design values shown are for new construction of local roads that are off-system routes not functionally classified at a higher classification. Projects to improve existing local roads differ from new construction in that the performance of the existing local road can guide the selection of design values.

The following subsections provide additional discussion for each design element and factors that may influence these values.

The design value thresholds for design exceptions and design waivers are presented in Appendix A and B respectively.

Table 5-1: Target Design Values for Local Roads

		Design Element	Rural	Rural Town	Suburban	Urban	Urban Core	Reference
Roadway	★	Design Speed [1]	40 to 50 MPH	25 to 30 MPH	40 MPH	25 to 30 MPH	20 to 25 MPH	See 5.1.3
	★	Lane Width	12-ft	11 to 12-ft	12-ft	11-ft	11-ft	See 5.1.6
	★	Shoulder Width (Uncurbed)	8-ft	4-ft	10-ft	8-ft	2-ft	See 5.1.7
		Offset to Face of Curb	N/A	1-ft	2-ft	2-ft	2-ft	See 4.10.12
		Curb Parking Lane Width[2]	N/A	9-ft	N/A	9-ft	9-ft	See 4.10.16
		Bike Lane Width	See 18.4					
		Speed Change Lane Width	12-ft	11 to 12-ft	12-ft	11-ft	11-ft	See 4.10.2
		Median Width	See 4.10.15					
	★	Horizontal Curve Radius (Minimum)	See 4.7					
	★	Cross Slope on a Tangent	2%					See 4.10.4
	★	Cross Slope on a Tangent (Maximum)	3%					See 4.10.4
	★	Superelevation Rate	See Table 4-3 through Table 4-7					See 4.7.3
		Minimum Grade (PGL)	0.25% (lined ditch channels), 0.3% (curbed facilities), or 0.5% (unpaved ditches)					See 4.8.1
	★	Maximum Grade (Level)	6%	8%	8%	8%	8%	See 4.8.1
	★	Maximum Grade (Rolling)	8%	8%	8%	8%	8%	See 4.8.1
	★	Vertical Clearance	16.5-ft					See 4.8.6
	★	Design Loading Structural Capacity	HL-93					AASHTO LRFD Bridge Design Specifications
	★	Stopping Sight Distance (SSD)	See 4.11.1					
		Design Vehicle	S-BUS-40					See 4.3.2
		Level of Service (LOS Design Year)	See 5.1.5					See 4.3.4.1

		Design Element	Rural	Rural Town	Suburban	Urban	Urban Core	Reference
Roadside		Clear Zone (Curbed)	See 4.10.7					
		Clear Zone (Uncurbed)	See 4.10.7					
		Sidewalk Width	N/A	6 to 8-ft	6 to 8-ft	6 to 8-ft	6 to 8-ft	See 19.2.2
		Sidewalk Offset	N/A	Varies				See 19.3.2
		Border Width[3]	N/A	10-ft	10-ft	10-ft	10-ft	See 4.10.9
Notes	★	Denotes that this is a design exception condition if the specified values in Appendix A are not met.						
		Denotes that this is a design waiver condition if the specified values in Appendix B are not met.						
	1	The selected Design Speed should meet the anticipated operating speed of the facility during non-peak hours.						
	2	If a bike lane is adjacent to a parking lane, the parking lane width should be increased to 12-ft to allow car doors to be open without bicyclist conflict. See Section 4.10 for additional guidance.						
	3	A 2-ft border width may be used where available ROW is limited or too costly.						

5.1.3 Design Speed

The design speed for local roads should reflect the appropriate environmental and terrain conditions for the various context classifications. The type and extent of adjacent development will generally suggest lower design speeds for these types of roadways due to the availability of ROW. Minimum design speeds for local roads are shown in **Appendix A**.

See **Section 4.2** for additional guidance on selecting the appropriate design speed.

When determining applicable radii and superelevation for design speeds see Table 4-3 for a summary of superelevation methodologies to be used.

5.1.3.1 Rural

Design speeds in the rural context may range from 40 mph (rolling terrain) to 50 mph (level terrain) for roads with greater than 400 VPD. For rural local roads with less than 400 VPD, 30 mph (rolling terrain) or 40 mph (level terrain) design speeds may be used.

Lower design speeds are applicable to roads with winding alignments in rolling terrain or where environmental conditions dictate. Higher design speeds are applicable to roads in level terrain or where environmental conditions are favorable.

5.1.3.2 Suburban

Design speeds for local roads in the suburban context range should ideally be 40 mph and will typically act as a transition segment between rural and urban or rural town sections of roadway.

5.1.3.3 Rural Town, Urban and Urban Core

Design speeds in these contexts are generally limited because of closely spaced intersections in a rural town or urban street grid. Since the function of local roads is to provide access to adjacent property in these contexts, **all design elements should be consistent with the character of activity adjacent to the street**. Design speeds may be reduced to be as low as 20 mph for urban core and design elements should encourage speeds to be no greater than 30 mph.

5.1.4 Design Traffic Volumes

Traffic volume is a major factor for local roads serving industrial or commercial areas. Local roads should be designed to accommodate traffic projections for a 20-year period into the future for all contexts. The ADT projected to some future design year should be the design basis. For additional guidance on the selection of the appropriate periods for forecasting design traffic volumes, see **Section 4.3.1** and **AASHTO's A Policy on Geometric Design of Highways and Streets**.

5.1.5 Levels of Service

The highest level of service practical and consistent with the anticipated conditions should be provided for local roads.

See **Section 4.3.4.1** for more information regarding level of service as it relates to facility design.

5.1.6 Travel Lane Width and Number of Lanes

Travel lane widths may vary from 10 to 12-ft depending on the context classification of the facility and the future ADT.

Minimum width of new or widened structures should accommodate the approach roadway width including shoulders.

5.1.6.1 Rural

Rural contexts should generally provide 12-ft lanes, especially on roads where the ADT > 2000 and substantial truck volumes are present or agricultural equipment frequently uses the road. For roadways with ADT between 400 and 2,000 and a design speed ≤ 50 mph a minimum of 11-ft lanes must be used. On very low-volume roadways (lower than 400 VPD), 10-ft lanes may be used.

Two travel lanes should be provided on local roads in rural areas. Additional lanes may be needed based on operational analysis.

5.1.6.2 Rural Town, Suburban, Urban and Urban Core

Rural town, suburban, urban, and urban core contexts commonly use 11 to 12-ft lane widths. In industrial areas, 12-ft lane widths should be provided. Industrial areas with constrained ROW may use 11-ft lane widths. In non-industrial areas with constrained ROW, 10-ft lane width may be used. 9-ft minimum lane width may be used in residential areas.

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for guidance on number of lane widths in these contexts.

5.1.7 Shoulders

Local roads shoulder widths may vary from 2 to 8-ft depending on the context classification of the facility. Where right turn lanes are present on uncurbed facilities, a 4-ft fully surfaced shoulder must be provided.

Facilities must be assessed to determine if bicycle accommodation are required on the shoulders. **Bike accessible shoulders must be fully surfaced for the entire shoulder width.**

See Chapter 18 for additional guidance on bicycle accommodations.

See **Section 4.10.3** for additional information on shoulders and shoulder surfacing requirements.

See **Chapter 12** for specific off-system bridge requirements for current ADT of 400 VPD or less.

A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated.

5.1.7.1 Rural

Outside shoulder widths on rural local roads are normally 4 to 8-ft. For two-lane rural roads, shoulder widths may be reduced to 2-ft if future ADT volumes are less than 400 VPD.

Mailbox turnouts should be considered on local roads with 400 or more VPD. Refer to TxDOT standard drawings for details regarding the design and construction of mailbox turnouts.

5.1.7.2 Suburban

Uncurbed shoulders in the suburban context can range from 4 to 10-ft wide.

5.1.7.3 Rural Town, Urban and Urban Core

Uncurbed shoulders in rural town, suburban, urban, and urban core contexts may vary from 2 to 8-ft shoulder widths.

Parking lanes in rural town, urban and urban core contexts may be used instead of shoulders. See Section 4.10.16 for guidance on the use of parking lanes.

5.1.8 Speed Change Lanes

The width of a speed change lane should match the through lane width of the facility. See **Section 4.10.2** and **AASHTO's A Policy on Geometric Design of Highways and Streets** for further discussion on speed change lane widths and their use.

5.1.9 Parking Lanes

Guidance on Parking Lanes is presented in **Section 4.10.16**.

5.1.10 Intersections

Guidance on Intersections is presented in **Chapter 13**.

5.1.11 Medians

Guidance on medians is presented in **Section 4.10.15**.

5.1.12 Curbs

Guidance on Curbs and Curb Offsets is presented in **Section 4.10.12**.

5.1.13 Cross Slope and Superelevation

Guidance on Cross Slope and Superelevation is presented in **Sections 4.10.4** and **4.7**.

5.1.14 Grades

Guidance for maximum Grades for local roads is presented in **Section 4.8.1**.

5.1.15 Roadside Design

Local roads must meet the minimum clear zone requirements shown in **Table 4-20**. Additional guidance on roadside design is presented in **Section 4.10.11**.

5.1.16 Structures

Bridges, culverts, walls, tunnels, and other structures must be designed in accordance with the current **AASHTO LRFD Bridge Design Manual**. The design loading must be HL-93 design live load unless design for a special vehicle is specified or warranted.

See **Chapter 12** for specific off-system bridge requirements for current ADT of 400 VPD or less.

Refer to the latest **TxDOT Bridge Design Manual - LFRD** for additional design guidance for bridges on TxDOT facilities.

5.1.17 Vertical Clearance

Vertical clearance requirements are presented in **Section 4.8.6**.

5.1.18 Bicycle and Pedestrian Facilities

Guidance for Bicycle and Pedestrian Facilities are presented in **Chapters 18** and **19**, respectively. All projects should be assessed for pedestrian and bicycle accommodations that may be required. **For specific jurisdictional requirements confirm with local entity for additional design criteria.**

5.2 Rural Design Element Characteristics

Two-lane local roads are a major part of the road system in rural areas. These roads should be designed to accommodate the highest practical criteria compatible with traffic and topography.

5.3 Rural Town, Suburban, Urban and Urban Core Design Element Characteristics

The design of local roads in the rural town context are similar to the design of local roads in the suburban, urban and urban core contexts. Local roads generally have lower traffic volumes than collectors and arterials and place emphasis on serving adjacent developments. A flexible and balanced approach to accommodate all modes of transportation should be applied.

Design features of local roads in urban areas are constrained by two major design controls:

- Type and extent of urban development that often limits the available ROW; and
- Zoning or regulatory restrictions.

The major design controls for local roads are intended to provide efficient operations. **The designer should be aware of local ordinances and resolutions that affect certain design features.**

Chapter 6
Collectors (4R)

6.1 Design Considerations

6.1.1 Introduction

A collector roadway will usually serve moderate traffic volumes and provides a circulation link between local roadways and arterial highways. **Design criteria presented in this chapter apply to new construction and reconstruction projects** for collector highways on the State Highway System.

This chapter discusses the features and design criteria for collectors and includes the following sections and subsections:

6.1 Design Considerations

- 6.1.1 Introduction;
- 6.1.2 Target Design Values;
- 6.1.3 Design Speed;
- 6.1.4 Design Traffic Volumes;
- 6.1.5 Levels of Service;
- 6.1.6 Travel Lane Width and Number of Lanes;
- 6.1.7 Shoulders;
- 6.1.8 Speed Change Lanes;
- 6.1.9 Parking Lanes;
- 6.1.10 Intersections;
- 6.1.11 Medians;
- 6.1.12 Curbs;
- 6.1.13 Cross Slope and Superelevation;
- 6.1.14 Grades;
- 6.1.15 Roadside Design;
- 6.1.16 Structures;
- 6.1.17 Vertical Clearance;
- 6.1.18 Grade Separations, Interchanges, and Ramps;
- 6.1.19 Pedestrian and Bicycle Facilities; and
- 6.1.20 Access Control.

6.2 Rural Design Elements

- 6.2.1 General Characteristics; and
- 6.2.2 Typical Sections.

6.3 Rural Town Design Elements

- 6.3.1 General Characteristics; and
- 6.3.2 Typical Sections.

6.4 Suburban, Urban, and Urban Core Design Elements

- 6.4.1 General Characteristics; and
- 6.4.2 Typical Sections.

This chapter is organized with an introductory section on the general design considerations for collectors, followed by separate design discussions for local roads in rural, rural town, suburban, urban, and urban core contexts.

6.1.2 Target Design Values

Table 6-1 shows target design values for Collectors for each context classification. These design values may be adjusted based on the anticipated target speed and a performance-based practical design evaluation of the corridor as discussed in **Chapter 2**.

The subsections following **Table 6-1** provide additional discussion for each design element and factors that may influence these values.

The design value thresholds for design exceptions and design waivers are presented in Appendix A and B respectively.

Table 6-1: Target Design Values for Collectors

Design Element			Rural	Rural Town	Suburban	Urban	Urban Core	Reference
Roadway	★	Design Speed[1]	50 to 70 MPH	35 to 40 MPH	45 to 55 MPH	35 to 40 MPH	25 to 30 MPH	See 6.1.3
	★	Travel Lane Width	12-ft	11 to 12-ft	12-ft	11-ft	11-ft	See 6.1.6
	★	Shoulder Width (Uncurbed)[2]	10-ft[2]	3 to 8-ft[2]	8-ft[2]	3 to 8-ft[2]	3 to 8-ft[2]	See 6.1.7
		Offset to Face of Curb	See 4.10.12	2-ft	See 4.10.12	2-ft	2-ft	See 4.10.12
		Curb Parking Lane Width[5]	N/A	10-ft	N/A	10-ft	10-ft	See 4.10.16
	★	Bike Lane Width	N/A	5-ft	N/A	5-ft	5-ft	See 18.4
		Speed Change Lane Width	12-ft	12-ft	12-ft	11-ft	11-ft	See 4.10.2
		Median Width	See 4.10.15					
	★	Horizontal Curve Radius	See 4.7					
	★	Cross Slope on a Tangent	2%					See 4.10.4
	★	Cross Slope on a Tangent (Maximum)	3%					See 4.10.4
	★	Superelevation Rate	See Table 4-3 through Table 4-7					See 4.7.3
		Minimum Grade (PGL)	0.25%(lined ditch channels), 0.3% (curbed facilities), or 0.5%(unpaved ditches)					See 4.8.1
	★	Maximum Grade (Level)	4%	8%	6%	8%	9%	See 4.8.1
	★	Maximum Grade (Rolling)	5%	9%	7%	9%	11%	See 4.8.1
	★	Vertical Clearance at New Structures[3]	16.5-ft[3]					See 4.8.6

Design Element			Rural	Rural Town	Suburban	Urban	Urban Core	Reference
	★	Design Loading Structural Capacity	HL-93					AASHTO LRFD Bridge Design Specs
	★	Stopping Sight Distance (SSD)	See 4.11.1					
		Design Vehicle[4]	WB-67	WB-67	WB-67	SU-40[4]	CITY-BUS[4]	See 4.3.2
		Level of Service (LOS Design Year)	B	B	B	B	C	See 4.3.4.1
Roadside		Clear Zone (Curbed)	See 4.10.7					
		Clear Zone (Uncurbed)	See 4.10.7					
		Sidewalk Width	5-ft (if applicable)	5-ft	5-ft	6 to 8-ft	5 to 8-ft	See 19.3.2
		Sidewalk Offset	N/A	Varies 6-ft or more for curbed roadways Clear Zone width or more for uncurbed roadways				See 19.3.2
		Border Width	N/A	20-ft	20-ft	20-ft	15-ft	See 4.10.9
Notes	★	Denotes that this is a design exception condition if the specified values in Appendix A are not met.						
		Denotes that this is a design waiver condition if the specified values in Appendix B are not met.						
	1	The selected Design Speed should meet the anticipated target speed of the facility during non-peak hours.						
	2	A 12-ft paved shoulder should be considered where the volume of truck traffic exceeds 250 DDHV. An 8-ft minimum paved shoulder should be considered for truck traffic volumes that are less than 250 and the highway is located on the Texas Highway Freight Network (THFN).						
	3	18.5-ft vertical clearance is required for highways on the Texas Highway Freight Network (THFN). Specific railroad company guidelines may require additional vertical clearance requirements.						
	4	Consider providing intersection turning radii for a WB-67 design vehicle on roadways or streets that are located on the Texas Freight Network and where high (> 10%) truck turning movements are known or expected.						
	5	If a bike lane is adjacent to a parking lane, the parking lane width should be increased to 12-ft to allow car doors to be open without bicyclist conflict. See Section 4.10 for additional guidance.						

6.1.3 Design Speed

The design speed for collectors can vary greatly between the five contexts. **The design speed should reflect the anticipated target speed during non-peak hours.** However, the design speed should not exceed the limits of prudent construction, ROW, and socioeconomic costs. Minimum design speeds for collectors are shown in **Appendix A**.

See **Section 4.2** for additional guidance on selecting the appropriate design speed.

When determining applicable radii and superelevation for design speeds see Table 4-3 for a summary of superelevation methodologies to be used.

6.1.3.1 Rural

Design speeds for rural collectors may be reduced to 40 mph or higher in rolling terrain and 50 mph or higher in level terrain for facilities with an ADT < 2,000 VPD. See **Section 4.6** for guidance on different types of terrain.

6.1.3.2 Rural Town

Design speeds for collectors in the rural town context may vary from 30 to 45 mph.

6.1.3.3 Suburban

Design speeds for collectors in the suburban context range from 35 to 60 mph and will typically act as a transition segment between rural and urban or rural town sections of roadway.

6.1.3.4 Urban and Urban Core

The design speed for urban collectors should range from 30 to 45 mph. The design speed for urban core collectors should range from 25 to 30 mph.

6.1.4 Design Traffic Volumes

Collector roadways for all contexts should be designed to accommodate traffic projections for a 20-year period into the future. However, some elements may be based on shorter or longer design periods (e.g., 20-year period for flexible pavements and 30-year period for rigid pavements). For additional guidance on the selection of the appropriate periods for forecasting design traffic volumes, see **Section 4.3.1** or **AASHTO's A Policy on Geometric Design of Highways and Streets**.

Projected volumes for other modes of transportation, such as pedestrians and or bicyclists, must also be estimated in the design of the facility. While future estimates of transit and non-motorized use may not be available, the designer may use available planning and land use documents to assist in determining future levels of nonmotorized demand.

6.1.5 Levels of Service

Collectors and their auxiliary facilities should be designed for Level of Service C or better in the design year for all contexts. Level of Service D may be allowable for heavily developed or constrained areas.

See **Section 4.3.4.1** for more information regarding level of service as it relates to facility design.

6.1.6 Travel Lane Width and Number of Lanes

Travel lane widths may vary from 10 to 12-ft depending on the context classification of the facility and anticipated target speed.

12-ft travel lanes are recommended on all roadways that are part of the Texas Highway Freight Network (THFN).

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 **HCM**. **The typical range of lanes for collectors is one to two travel lanes in each direction.**

6.1.6.1 Rural

Rural collectors should be designed to 12-ft due to the higher operating speeds usually associated with these types of facilities.

11-ft lanes may be used on two lane collectors that:

- Have an operating speed of 45 MPH or less and a future ADT less than 2,000 VPD;
- Have an operating speed of 60 MPH or higher and a future ADT less than 400 VPD; or
- Have an operating speed of 70 MPH or less and a future ADT less than 1,500.

10-ft lanes may be used on two lane collectors that:

- Have operating speeds of 55 MPH or less and future ADT's less than 400 VPD.

6.1.6.2 Rural Town

Rural town collectors should provide 12-ft lanes due to the agricultural vehicles and trucks that will drive on these types of facilities. 11-ft lanes may be used in restrictive ROW conditions.

Lane widths may be reduced to 10-ft in highly constrained areas where both the following conditions are present:

- Truck and bus volumes are low (less than 5%); and
- The anticipated operating speed is less than or equal to 35 mph.

Engineering judgement must be exercised when determining if a 10-ft lane width is acceptable. **Use of 10-ft travel lanes must be approved by the District and approval documented in the project file.**

6.1.6.3 Suburban, Urban, Urban Core

Suburban, urban, and urban core collectors will commonly use 11-ft lanes on facilities with lower operating speeds as a traffic calming measure (i.e., 45 mph or less). If the anticipated target speed is 50 mph or greater and or truck volumes are significant (>10%), 12-ft lanes should be used.

Lane widths may be reduced to 10-ft in highly constrained areas where the conditions described in 6.1.6.2 are met.

6.1.7 Shoulders

Collector shoulder widths may vary from 2 to 10-ft depending on the context classification, ADT, and anticipated target speed of the facility.

A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated.

See **Section 4.10.3** for additional information on shoulders and shoulder surfacing requirements.

6.1.7.1 Rural

Outside shoulder widths on rural collectors can vary from 2 to 10-ft as outlined in the subsequent section. **Rural collectors on the Texas Highway Freight Network (THFN) should provide 10-ft minimum outside shoulders.**

6.1.7.1.1 Two-Lane Rural Collectors

The minimum outside shoulder must be 10-ft for all design speeds and the future ADT is greater than 2,000 VPD. The outside shoulder width may be reduced to a minimum of 8-ft in highly constrained situations for any ADT or if the future ADT is between 1,500–2,000 VPD. A minimum 6-ft shoulder must be provided if the future ADT is between 400–1,500 VPD and the design speed is 50 mph or greater. The outside shoulder width may be reduced to a minimum 4-ft if the future ADT is between 400–1,500 VPD and the design speed is 45 mph or less. A minimum of 2-ft outside shoulders may be used if future ADT volumes are less than 400 VPD. **4-ft minimum shoulders must be used at locations adjacent to roadside barrier.**

6.1.7.1.2 Multi-Lane Rural Collectors

Outside shoulder widths may be reduced to 8-ft for highly constrained situations for both divided and undivided facilities.

A minimum of 4-ft inside shoulders must be provided for 4 lane divided rural collectors. On 6 lane or more divided collectors, 10-ft inside shoulders should be provided. The inside shoulder width may be reduced to 4-ft in constrained situations.

When right or left turn lanes are present, a 4-ft minimum shoulder must be provided for the turn lane.

6.1.7.2 Suburban

Suburban collector shoulder widths can vary from 4 to 10-ft. It is preferred to have 8 to 10-ft shoulder widths on suburban roadways with high operating speeds (50 mph or greater).

When right or left turn lanes are present, a 4-ft minimum shoulder must be provided for the turn lane.

Due to the higher speeds frequently associated with this context, shoulders are usually provided instead of curbs. See **Section 4.10.12** for additional information on curbs.

6.1.7.3 Rural Town, Urban, and Urban Core

Rural town, urban, and urban core contexts will typically use curbs and maintain curb offsets instead of shoulders due to the lower operating speeds. See **Section 4.10.12** for curb offset requirements and additional information on curbs.

For uncurbed roadways, shoulder widths may vary from 3 to 8-ft. **Uncurbed collectors on the Texas Highway Freight Network (THFN) should provide at least 8 ft shoulders for these contexts.**

6.1.8 Speed Change Lanes

Collectors generally incorporate the following types of speed change lanes:

- Left and right turn lanes;
- Passing or climbing lanes; and
- Acceleration and deceleration lanes for entrance and exit ramps.

Speed change lanes should be provided where vehicles must slow to leave a facility or accelerate to merge onto a facility.

The width of a speed change lane should match the through lane width of the facility. See **Section 4.10.2** and **AASHTO's A Policy on Geometric Design of Highways and Streets** for further discussion on speed change lane widths and their use.

6.1.9 Parking Lanes

Guidance on Parking Lanes is presented in **Section 4.10.16**.

6.1.10 Intersections

Guidance on Intersections is presented in **Chapter 13**.

6.1.11 Medians

Guidance on Medians is presented in **Section 4.10.15**.

6.1.12 Curbs

Guidance on Curbs and Curb Offsets is presented in **Section 4.10.12**.

6.1.13 Cross Slope and Superelevation

Guidance on Cross Slope and Superelevation is presented in **Section 4.10.4** and **4.7**.

6.1.14 Grades

Guidance for maximum Grades for local roads is presented in **Section 4.8.1**.

6.1.15 Roadside Design

Collectors must meet the minimum clear zone requirements shown in **Table 4-20**. Additional guidance on roadside design is presented in **Section 4.10.11**.

Guidance for lateral offset is presented in **Section 4.10.8**.

6.1.16 Structures

Bridges, culverts, walls, tunnels, and other structures must be designed in accordance with the current **AASHTO LRFD Bridge Design Manual**. The design loading must be HL-93 design live load unless design for a special vehicle is specified or warranted.

Refer to the latest **TxDOT Bridge Design Manual - LRFD** for additional design guidance for bridges on TxDOT facilities.

6.1.17 Vertical Clearance

Vertical clearance requirements are presented in **Section 4.8.6**.

6.1.18 Grade Separations, Interchanges, and Ramps

Guidance for grade separations, interchanges, and ramps are presented in **Chapter 15**.

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional information on grade separations, interchanges, and ramps.

6.1.19 Bicycle and Pedestrian Facilities

Guidance for bicycle and pedestrian facilities is presented in **Chapters 18** and **19**, respectively. All projects should be assessed for pedestrian and bicycle accommodations that may be required.

6.1.20 Access Control

The installation of driveways and all access points along collector highways must be in accordance with TxDOT's **AMM**.

6.2 Rural Design Elements

6.2.1 General Characteristics

Two-lane collectors are an important part of the rural highway system in that they provide connectivity between local roads and streets with arterials. They should be designed with the most favorable horizontal alignment, profile, and cross section practical that is consistent with traffic volume and topography.

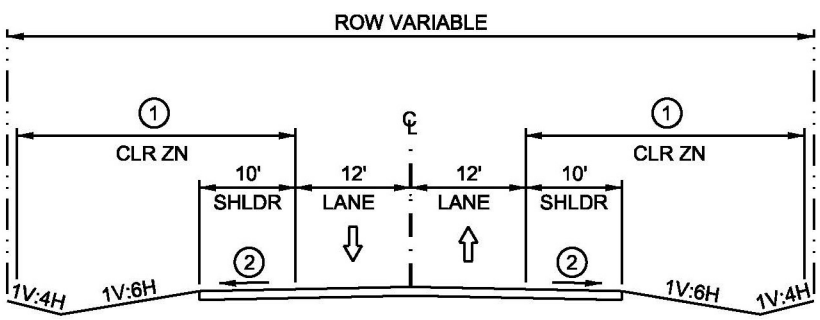
The basic information needed for the design of rural collectors include:

- Crash History;
- Current and Projected Traffic Volumes;
- Terrain; and
- Horizontal and vertical alignment.

6.2.2 Typical Sections

Rural collector example typical sections are shown in **Figure 6-1** through **Figure 6-5** below. The geometric dimensions shown in these sections represent examples of different conditions and may be adjusted in accordance with **Section 6.1**.

Selection of the appropriate cross-section is important for optimal safety and operational performance. See **Sections 7.2.2** and **7.2.5** in the Arterials Chapter for additional guidelines on selecting cross-sections for rural highways.



1 SEE SECTION 4.10.7
2 SEE SECTION 4.10.4

Figure 6-1: Rural Typical Section – 2 Lane

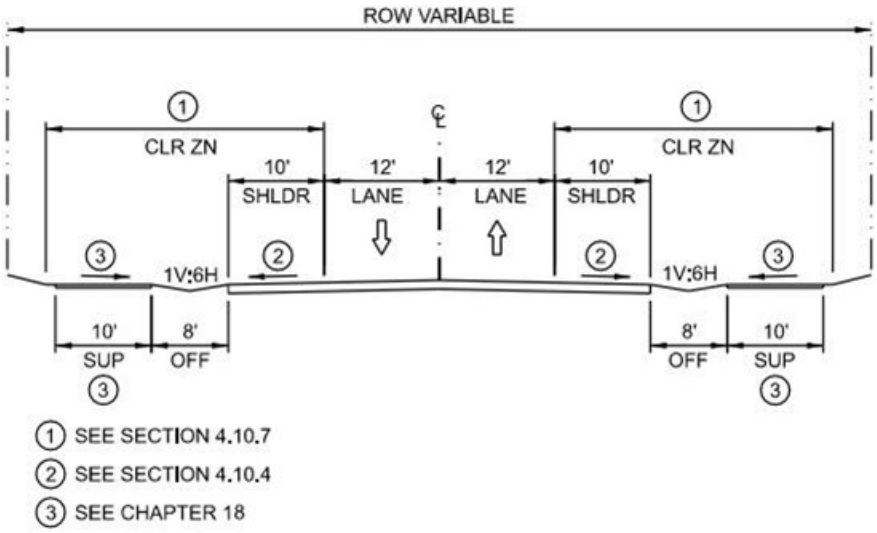


Figure 6-2: Rural Typical Section – 2 Lanes with SUP

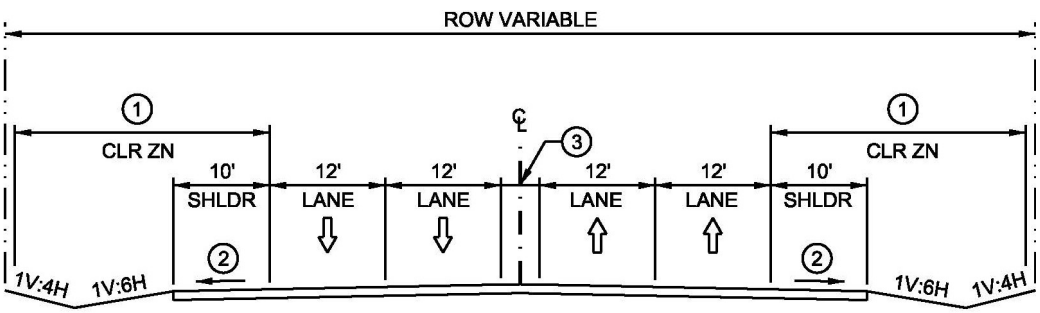
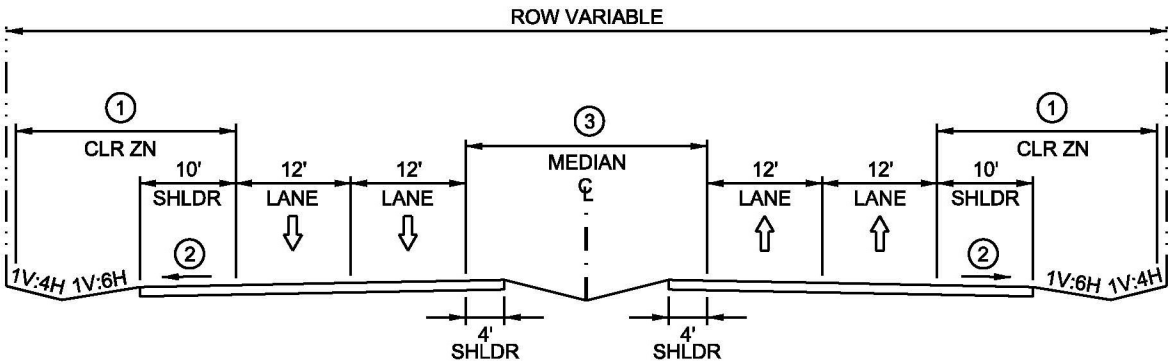
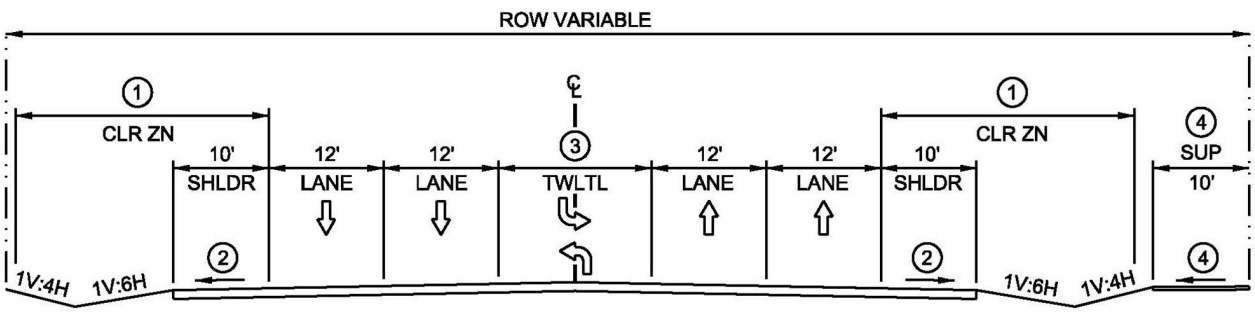


Figure 6-3: Rural Typical Section – Multilane Undivided w/ Median Buffer



1. SEE SECTION 4.10.7
2. SEE SECTION 4.10.4
3. SEE SECTION 4.10.15

Figure 6-4: Rural Typical Section – Multilane Divided (4 Lanes)



1. SEE SECTION 4.10.7
2. SEE SECTION 4.10.4
3. SEE SECTION 4.10.15.4
4. SEE CHAPTER 18

Figure 6-5: Rural Typical Section – Multilane Undivided with TWLTL

6.3 Rural Town Design Elements

6.3.1 General Characteristics

The rural town context applies to roads in rural areas located within a developed community. Collectors in the rural town context will generally have design speeds of 40 mph or less. The design speeds may be decreased in stages as drivers leave the rural environment and approach the center of a rural town. On-street parking for rural town collectors should be considered in central portions of a rural town where businesses are commonly located. Additionally, pedestrian and bicyclist flows may increase within a rural town creating the need for pedestrian and bicycle facility accommodations.

Rural town environments may differ in their operating speeds, needs for parking, pedestrian, and bicycle facilities, just like suburban, urban, and urban core contexts may also differ. Flexibility in the development of design features may be appropriate to meet the varying needs of rural towns.

6.3.2 Typical Sections

Rural town collector example typical sections are shown in **Figure 6-6** and **Figure 6-7** below. The geometric dimensions shown in these sections may be adjusted in accordance with **Section 6.1**.

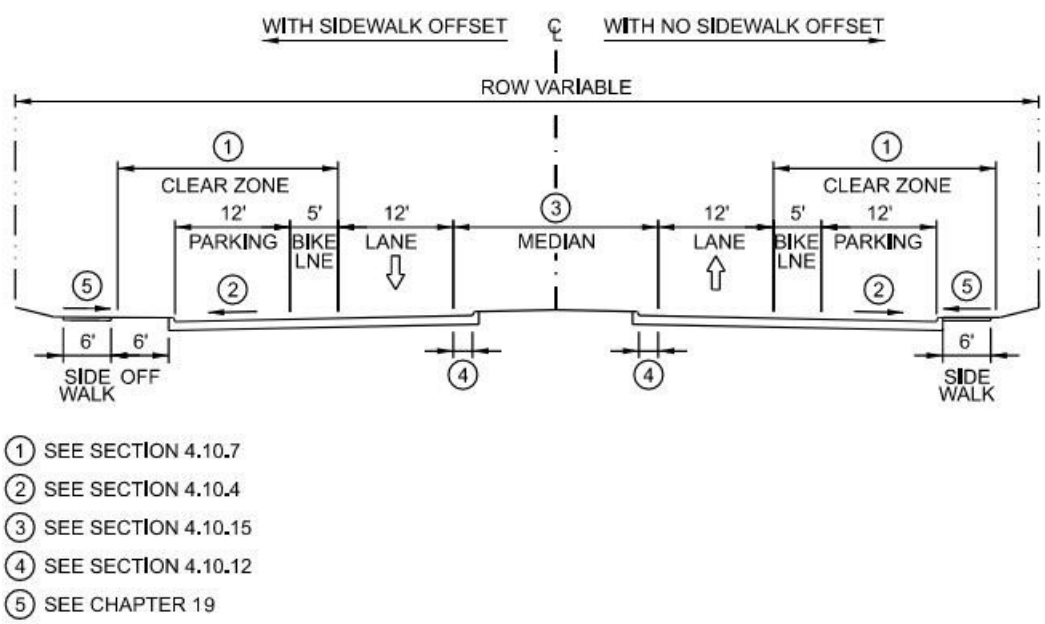


Figure 6-6: Rural Town Typical Section – Raised Median with Parking Lane

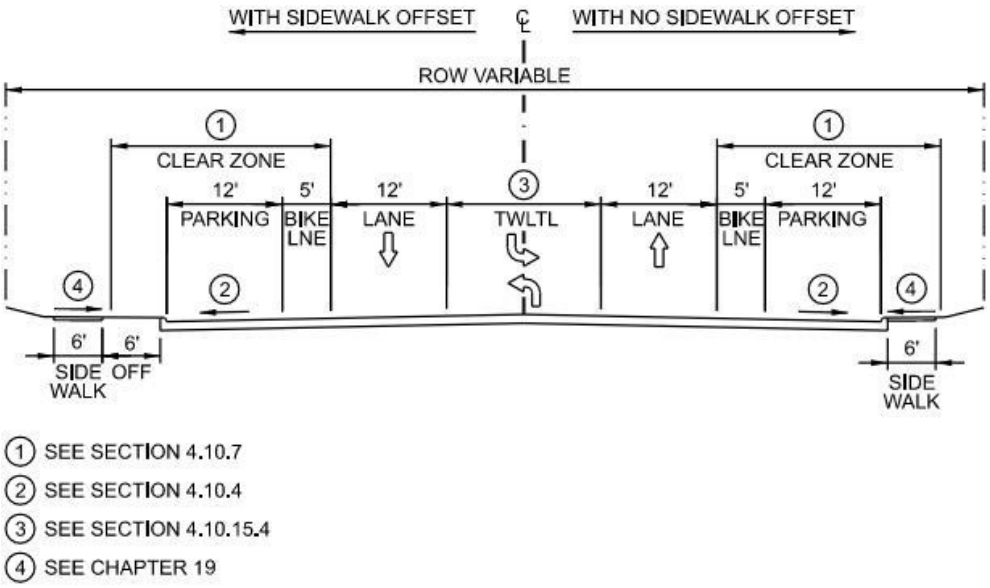


Figure 6-7: Rural Town Typical Section – Two-Way Left Turn Lane (TWLTL) with Parking Lane

6.4 Suburban, Urban, and Urban Core Design Elements

6.4.1 General Characteristics

Collectors in the urban areas are like the rural town context in that they are designed with a flexible approach to meet the needs of suburban, urban, and urban core contexts. As a collector road moves from the suburban context to the urban context, and then to the urban core context, the emphasis on maintaining higher operating speeds decreases and the importance of providing on street parking, pedestrian, and bicycle accommodations increases.

A flexible and balanced design approach to serve all transportation modes appropriately should be applied. The balance among transportation modes may differ between projects based on demand for each transportation mode and established corridor plans.

The function of collectors in suburban, urban, and urban core areas is equally divided between mobility and access. When a major objective of the design is to expedite traffic mobility, there is additional criteria that should be evaluated:

- Minimizing conflict points;
- Providing adequate storage length for all turning movements;
- Minimizing conflicts with pedestrians and bicyclists;
- Coordinating driveways on opposite sides of the road;
- Locating signals to meet progression needs; and
- Maintaining efficient circulation while providing adequate ingress and egress capacity.

6.4.2 Typical Sections

Suburban, Urban, and Urban Core example typical sections are shown in **Figure 6-8** through **Figure 6-15** below. The geometric dimensions shown in these sections may be adjusted in accordance with **Section 6.1**.

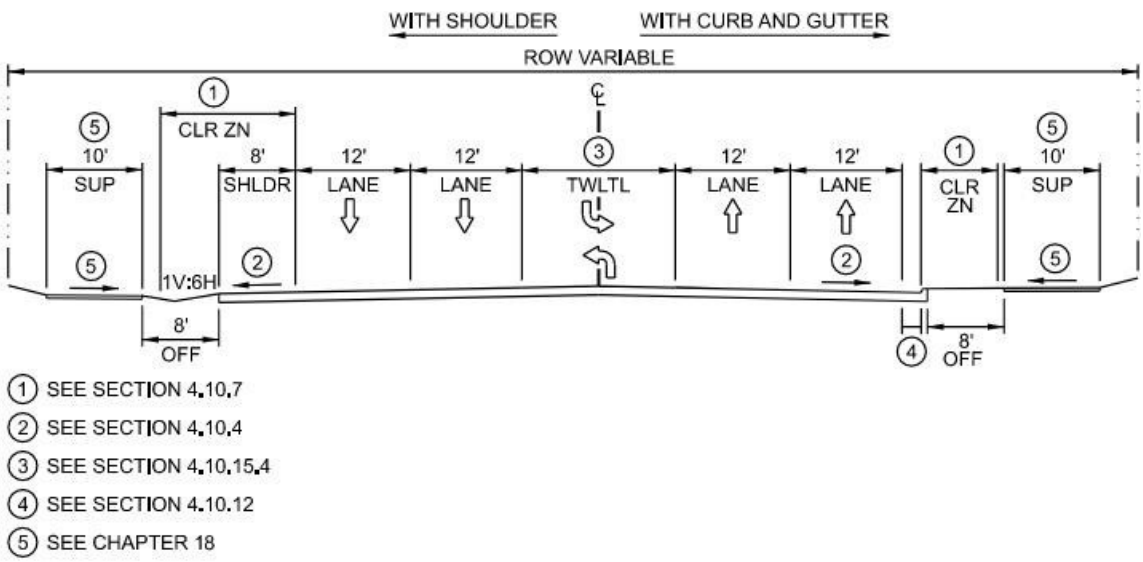


Figure 6-8: Suburban Typical Section – Two-Way Left Turn Lane (TWLTL)

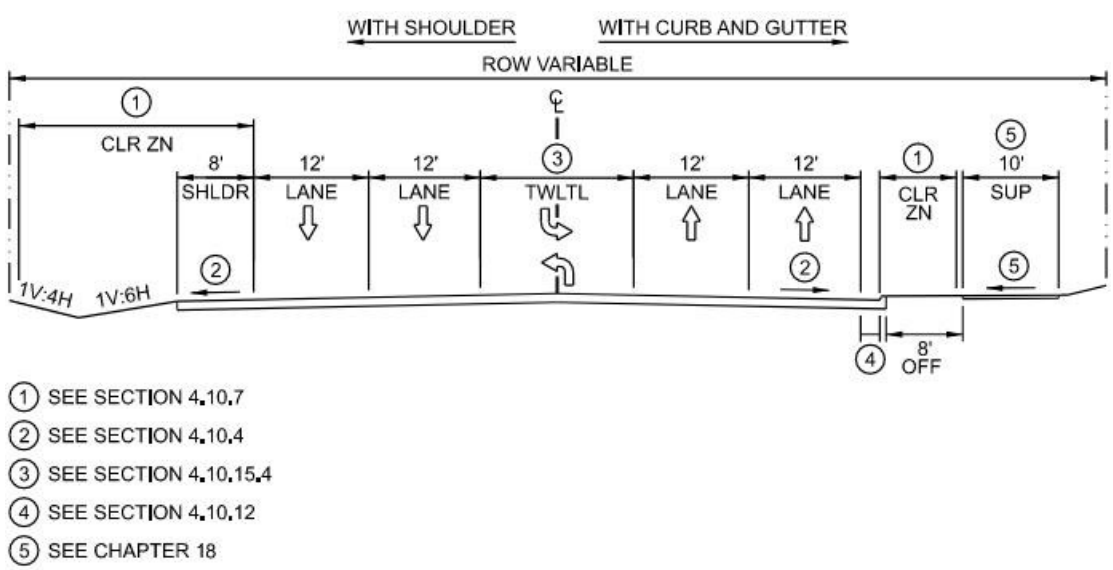
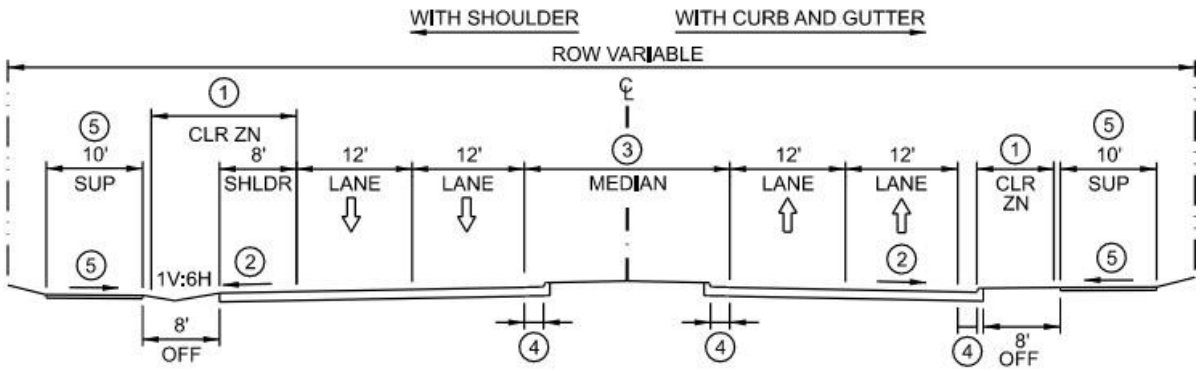
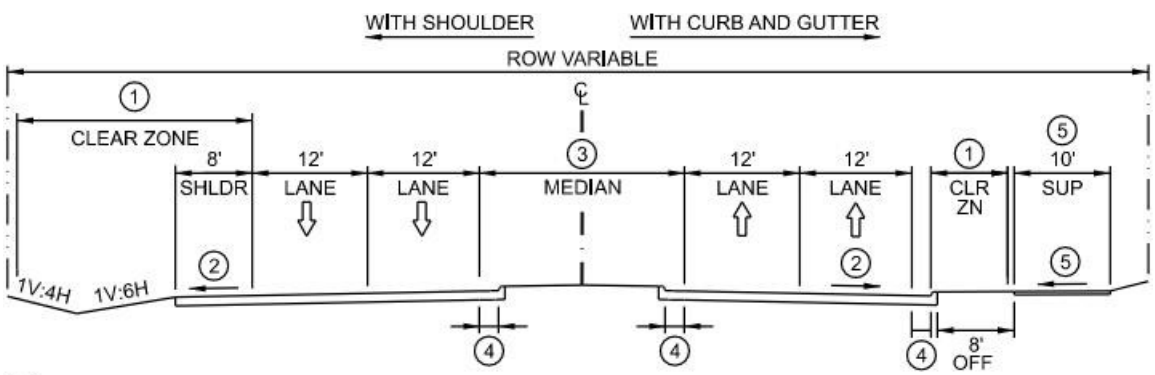


Figure 6-9: Suburban Typical Section – Two-Way Left Turn Lane (TWLTL) with Constrained ROW



① SEE SECTION 4.10.7
② SEE SECTION 4.10.4
③ SEE SECTION 4.10.15
④ SEE SECTION 4.10.12
⑤ SEE CHAPTER 18

Figure 6-10: Suburban Typical Section – Raised Median



① SEE SECTION 4.10.7
② SEE SECTION 4.10.4
③ SEE SECTION 4.10.15
④ SEE SECTION 4.10.12
⑤ SEE CHAPTER 18

Figure 6-11: Suburban Typical Section – Raised Median with Constrained ROW

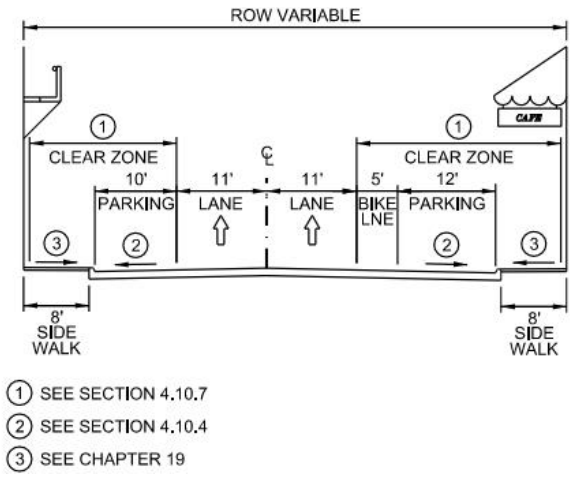


Figure 6-12: Urban Core Typical Section (One-Way)

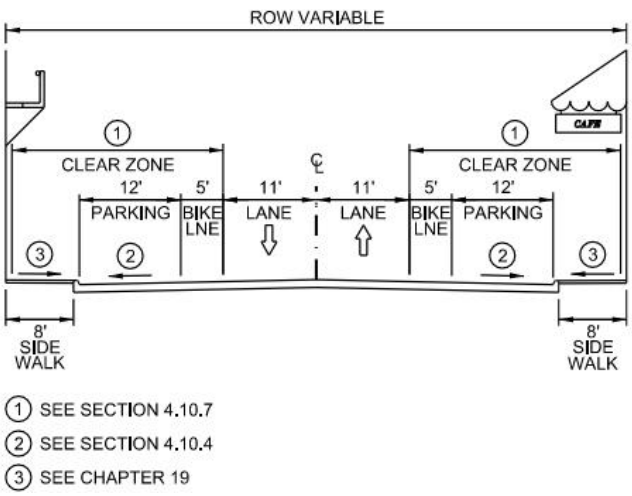
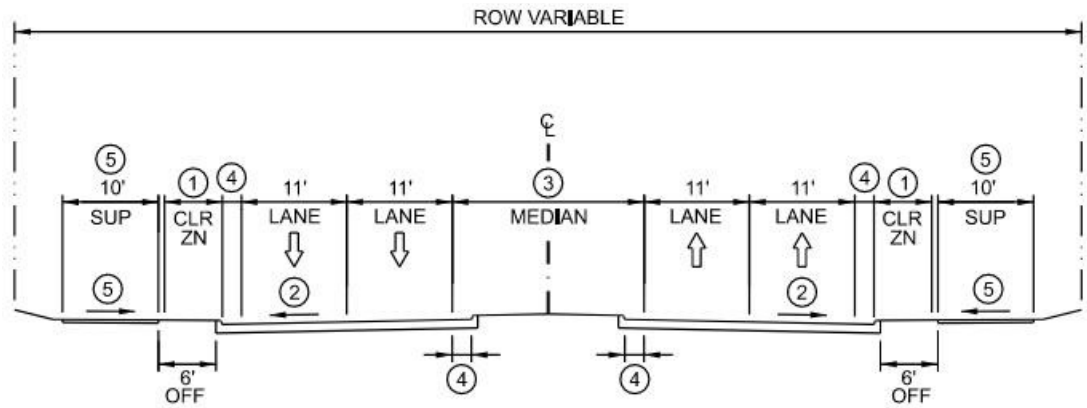
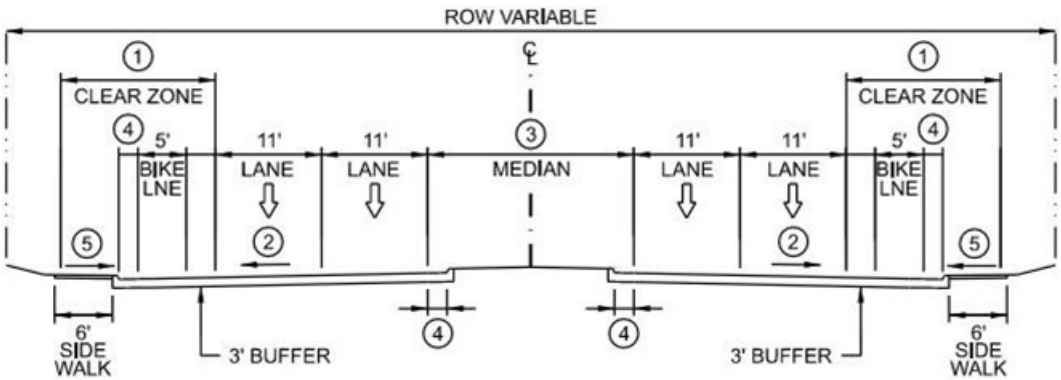


Figure 6-13: Urban Core Typical Section (Two-Way)



1 SEE SECTION 4.10.7
2 SEE SECTION 4.10.4
3 SEE SECTION 4.10.15
4 SEE SECTION 4.10.12
5 SEE CHAPTER 18

Figure 6-14: Urban Typical Section – Separated Shared Use Path (SUP)



1 SEE SECTION 4.10.7
2 SEE SECTION 4.10.4
3 SEE SECTION 4.10.15
4 SEE SECTION 4.10.12
5 SEE CHAPTER 19

Figure 6-15: Urban Typical Section – Bike Lane

Chapter 7
Arterials (4R)

7.1 Design Considerations

7.1.1 Introduction

Arterials provide for travel between major points in both urban and rural areas. **Design criteria presented in this chapter apply to new construction and reconstruction projects** for arterial highways on the State Highway System.

Reconstruction work generally includes substantial changes in the geometric character of the highway, such as widening to provide additional travel lanes, significant horizontal or vertical realignments, major improvements to the pavement structure to improve long-term service, or bridge replacement. For further discussion of this definition refer to **AASHTO's A Policy of Geometric Design of Highways and Streets.**

This chapter discusses the features and design criteria for arterials and includes the following sections and subsections:

7.1 Design Considerations

- 7.1.1 Introduction;
- 7.1.2 Target Design Values;
- 7.1.3 Design Speed;
- 7.1.4 Design Traffic Volumes;
- 7.1.5 Levels of Service;
- 7.1.6 Travel Lane Width and Number of Lanes;
- 7.1.7 Shoulders;
- 7.1.8 Speed Change Lanes;
- 7.1.9 Parking Lanes;
- 7.1.10 Intersections;
- 7.1.11 Medians;
- 7.1.12 Curbs;
- 7.1.13 Cross Slope and Superelevation;
- 7.1.14 Grades;
- 7.1.15 Roadside Design;
- 7.1.16 Structures;
- 7.1.17 Vertical Clearance;
- 7.1.18 Grade Separations, Interchanges, and Ramps;
- 7.1.19 Pedestrian and Bicycle Facilities; and
- 7.1.20 Access Control.

7.2 Rural Design Elements

- 7.2.1 General Characteristics;
- 7.2.2 Typical Section;
- 7.2.3 Transitions to Four-Lane Divided Highways;
- 7.2.4 Converting Two-Lane Roadways to Four-Lane Divided Facilities; and
- 7.2.5 Converting Two-Lane Roadways to Optimized Roadway Cross-Sections.

7.3 Rural Town Design Elements

- 7.3.1 General Characteristics; and
- 7.3.2 Typical Section.

7.4 Suburban Design Elements

- 7.4.1 General Characteristics; and
- 7.4.2 Typical Section.

7.5 Urban and Urban Core Design Elements

- 7.5.1 General Characteristics; and
- 7.5.2 Typical Section

This chapter is organized with an introductory section on the general design considerations for arterials, followed by separate design discussions for local roads in rural, rural town, suburban, urban, and urban core contexts.

7.1.2 Target Design Values

Table 7-1 shows target design values for Arterials for each context classification. These design values may be adjusted based on the anticipated target speed and a performance-based practical design evaluation of the corridor (see **Chapter 2**).

The subsections following **Table 7-1** provide additional discussion for each design element and factors influencing these values.

The design value thresholds for design exceptions and design waivers are presented in Appendix A and Appendix B respectively.

Table 7-1: Target Design Values for Arterials

Design Element			Rural	Rural Town	Suburban	Urban	Urban Core	Reference
Roadway	★	Design Speed[1]	70 MPH	40 to 45 MPH	50 to 60 MPH	40 to 45 MPH	30 to 35 MPH	See 7.1.3
	★	Travel Lane Width	12-ft	11 to 12-ft	12-ft	11-ft	11-ft	See 7.1.6
	★	Shoulder Width (Uncurbed)[2]	10-ft[2]	4 to 10-ft[2]	8 to 10-ft[2]	4 to 10-ft[2]	4 to 10-ft[2]	See 7.1.7
		Offset to Face of Curb	2-ft					See 4.10.12
		Curb Parking Lane Width[5]	N/A	10-ft	N/A	10-ft	10-ft	See 4.10.16
		On-Street Bike Lane Width	N/A	5-ft	5-ft	5-ft	5-ft	See 18.4.5
		Speed Change Lane Width	12-ft	12-ft	12-ft	11-ft	11-ft	See 4.10.2
		Median Width	See 4.10.15					
	★	Horizontal Curve Radius	See 4.7					
	★	Cross Slope on a Tangent	2%					See 4.10.4
	★	Cross Slope on a Tangent (Maximum)	3%					See 4.10.4
	★	Superelevation Rate	See Table 4-3 through Table 4-7					See 4.7.3

		Design Element	Rural	Rural Town	Suburban	Urban	Urban Core	Reference
		Minimum Grade	0.25% (lined ditch channels), 0.3% (curbed facilities), or 0.5% (unpaved ditches)					See 4.8.1
	★	Maximum Grade (Level)	3%	6%	5%	6%	8%	See 4.8.1
	★	Maximum Grade (Rolling)	4%	7%	6%	7%	9%	See 4.8.1
	★	Vertical Clearance at New Structures[3]	18.5-ft[3]					See 4.8.6
	★	Design Loading Structural Capacity	HL-93					AASHTO LRFD Bridge Design Specs
	★	Stopping Sight Distance (SSD)	See 4.11.1					
		Design Vehicle[4]	WB-67	WB-67	WB-67	SU-40[4]	CITY-BUS[4]	See 4.3.2
		Level of Service (LOS Design Year)	B	B	B	B	C	See 4.3.4.1
Roadside	⚑	Clear Zone (Curbed)	See 4.10.7					
	⚑	Clear Zone (Uncurbed)	See 4.10.7					
		Sidewalk Width	5-ft (if applicable)	5-ft	5-ft	6 to 8-ft	5 to 8-ft	See 19.3.2
		Sidewalk Offset	N/A	Varies 6-ft or more for curbed roadways Clear Zone width or more for uncurbed roadways				See 19.3.2
		Border Width	N/A	20-ft	20-ft	20-ft	15-ft	See 4.10.9

Notes	
★	Denotes that this is a design exception condition if the specified values in Appendix A are not met.
⚑	Denotes that this is a design waiver condition if the specified values in Appendix B are not met.
1	The selected Design Speed should meet the anticipated target speed of the facility during non-peak hours.
2	A 12-ft paved shoulder should be considered where the volume of truck traffic exceeds 250 DDHV. An 8-ft minimum paved shoulder should be considered for truck traffic volumes that are less than 250 and the highway is located on the Texas Highway Freight Network (THFN).
3	A 16.5-ft vertical clearance may be used if the highway is not on the Texas Highway Freight Network (THFN). Specific railroad company guidelines may require additional vertical clearance requirements.
4	Consider providing intersection turning radii for a WB-67 design vehicle on roadways or streets that are located on the Texas Freight Network and when high (>10%) truck turning movements are known or expected.
5	If a bike lane is adjacent to a parking lane, the parking lane width should be increased to 12-ft to allow car doors to be open without bicyclist conflict. See Section 4.10 for additional guidance.

7.1.3 Design Speed

The design speed for arterials can vary greatly between the five contexts. **The design speed should reflect the anticipated target speed 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 arterials are shown in **Appendix A.**

See **Section 4.2.3** for additional guidance on selecting the appropriate design speed.

When determining applicable radii and superelevation for design speeds see Table 4-3 for a summary of superelevation methodologies to be used.

7.1.3.1 Rural

Design speeds for rural arterials must be equal to 60 mph or higher in rolling terrain and 70 mph or higher in level terrain. See **Section 4.6** for guidance on the different types of terrain.

7.1.3.2 Rural Town

Rural town arterial design speeds are similar to the urban context in that they may vary from 30 to 45 mph.

7.1.3.3 Suburban

Design speeds for arterials in the suburban context range from 40 to 60 mph and will typically act as a transition segment between rural and urban or rural town sections of roadway.

7.1.3.4 Urban and Urban Core

Urban core arterials generally have design speeds of 35 mph or less while urban arterials typically have design speeds ranging from 30 to 45 mph.

7.1.4 Design Traffic Volumes

Arterial roadways for all contexts should be designed to accommodate traffic projections for a 20-year period into the future. However, some elements may be based on shorter or longer design periods (e.g., a 20-year period for flexible pavements and a 30-year period for rigid pavements). For additional guidance on selecting the appropriate periods for forecasting design traffic volumes, see **Section 4.3.1** or refer to **AASHTO's A Policy on Geometric Design of Highways and Streets.**

Projected volumes for other modes of transportation, such as pedestrians and or bicyclists, must also be estimated in the design of the facility. While future estimates of transit and non-motorized modes may not be available, the designer may use available planning and land use documents to assist in determining future levels of nonmotorized demand.

7.1.5 Levels of Service

Arterials and their auxiliary facilities in the rural context should be designed for Level of Service B or better in the design year as defined in the **HCM**. All other contexts should be designed to Level of Service C or better. Level of Service D may be allowable for heavily developed or constrained areas.

See **Section 4.3.4.1** for more information regarding level of service as it relates to facility design.

7.1.5.1 Suburban, Urban and Urban Core

Arterials in dense developed urban areas may necessitate the use of Level of Service C or D in the design year.

The **HCM** describes methodology and applications for level-of-service criteria and capacity for automobiles, as well as pedestrians, bicycles, and transit, for urban street facilities. Urban street facilities as defined as having a length of:

- 1 mile or more in downtown areas; or
- Greater than 2 miles in other urbanized areas.

It also provides similar guidance for urban street segments, which are shorter than urban street facilities and allow analysis of more specific locations as needed. The theoretical automobile capacity of a through lane on an urban street facility or segment is 1,800 vph; adjustment factors reduce this capacity to account for the effects of certain roadway characteristics. Refer to the **HCM** methodology should be used to determine the recommended number of through lanes needed for given conditions on a specific facility or segment.

7.1.5.2 Rural Town

Where arterials in rural areas pass through a rural town context and non-motorized roadway users are present, or likely to be present in the future, the motor-vehicle level of service may be reduced to provide a more balanced level of service for all modes of transportation.

7.1.6 Travel Lane Width and Number of Lanes

Travel lane widths may vary from 10 to 12-ft depending on the context classification of the facility and anticipated target speed.

12-ft travel lanes are recommended on all roadways that are part of the Texas Highway Freight Network (THFN).

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 **HCM**. **The typical range of lanes for arterials is two to four travel lanes in each direction.**

7.1.6.1 Rural

Due to the high speeds associated with rural arterials, **lane widths must be designed to 12-ft**.

7.1.6.2 Rural Town

Rural town arterials should provide 12-ft lanes due to the agricultural vehicles and trucks that will drive on these types of facilities. 11-ft lanes may be used in restrictive Right of Way (ROW) conditions.

Lane widths may be reduced to 10-ft in highly constrained areas where both of the following conditions are present:

- Truck and bus volumes are low (less than 5%); and
- The anticipated operating speed is less than or equal to 40 mph.

Engineering judgement must be exercised when determining if a 10-ft lane width is acceptable. **Use of 10-ft travel lanes must be approved by the District and approval documented in the project file**.

7.1.6.3 Suburban, Urban and Urban Core

Suburban, urban, and urban core arterials will commonly use 11-ft lanes on facilities with lower target speeds as a traffic calming measure (i.e., 45 mph or less). If the anticipated target speed is 50 mph or greater and/or truck volumes are significant (>10%), 12-ft lanes should be used.

Lane widths may be reduced to 10-ft if the conditions described in Section 7.1.6.2 are met.

7.1.7 Shoulders

Arterial shoulder widths may vary from 4 to 10-ft depending on the context classification of the facility and anticipated target speed.

A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated.

See Section 4.10.3 for additional information on shoulders and shoulder surfacing requirements.

7.1.7.1 Rural

Outside shoulder widths on rural arterials should normally be 10-ft since they will typically have high operating speeds (60 mph or greater). In highly constrained situations, outside shoulder widths may be reduced to 8-ft. **Rural arterials on the Texas Highway Freight Network (THFN) should provide 10-ft minimum outside shoulders.**

7.1.7.1.1 Two-Lane Rural Arterials

For two-lane rural arterials, shoulder widths may be reduced to 4-ft if future ADT volumes are less than 400 VPD. **A minimum of 4-ft shoulders must be used at locations adjacent to roadside barrier**.

7.1.7.1.2 Multi-Lane Rural Arterials

A minimum inside shoulder width of 4-ft must be provided for 4-lane divided rural arterials. On 6-lane or more divided arterials, 10-ft inside shoulders should be provided. The inside shoulder width may be reduced to 4-ft in constrained situations.

When right or left turn lanes are present, a 4-ft minimum shoulder must be provided for the turn lane.

7.1.7.2 Suburban

Suburban arterial shoulder widths can vary from 4 to 10-ft. It is preferred to have 8 to 10-ft shoulder widths on suburban roadways with high operating speeds (50 mph or greater).

When right or left turn lanes are present, a 4-ft minimum shoulder must be provided for the turn lane.

Due to the higher speeds frequently associated with this context, shoulders are usually provided instead of curbs. See Section 4.10.12 for additional information on curbs.

7.1.7.3 Rural Town, Urban and Urban Core

Rural town, urban, and urban core contexts will typically use curbs and maintain curb offsets

instead of shoulders due to the lower operating speeds. See **Section 4.10.12** for curb offset requirements and additional information on curbs.

For uncurbed roadways, shoulder widths may vary from 4 to 10-ft. **Uncurbed arterials on the Texas Highway Freight Network (THFN) should provide at least 8 ft outside shoulders for these contexts.**

7.1.8 Speed Change Lanes

Arterials can incorporate all types of speed change lanes including:

- Left and right turn lanes;
- Passing or climbing lanes;
- Acceleration and deceleration lanes for entrance and exit ramps; and
- Weaving lanes.

Speed change lanes should be provided where vehicles must slow to leave a facility or accelerate to merge onto a facility.

The width of a speed change lane should match the through lane width of the facility. See **Section 4.10.2** and refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional guidance on speed change lane widths and their use.

If climbing lanes are warranted, for guidance on climbing lanes see **Section 4.10.2.12.**

7.1.9 Parking Lanes

For guidance on Parking Lanes, see **Section 4.10.16.**

7.1.10 Intersections

For guidance on Intersections, see **Chapter 13**.

7.1.11 Medians

For guidance on Medians, see **Section 4.10.15**.

7.1.12 Curbs

For guidance on Curbs and Curb Offsets, see **Section 4.10.12**.

7.1.13 Cross Slope and Superelevation

For guidance on Cross Slope and Superelevation, see **Sections 4.10.4** and **4.7**.

7.1.14 Grades

For guidance for maximum Grades for local roads, see **Section 4.8.1**.

7.1.15 Roadside Design

There are typically two primary considerations for roadside design along the traveled way for arterials – clear zones and lateral offset.

Lower-speed arterials in urban and urban core areas should provide a balanced roadway design for all users. These considerations may include providing increased access and mobility for nonmotorized modes (pedestrians, bicyclists, etc.), facilitating transit operations, and achieving other community goals.

7.1.15.1 Clear Zone

Arterials must meet the minimum clear zone requirements shown in **Table 4-20**. For additional guidance on roadside design see **Section 4.10.11**.

7.1.15.2 Lateral Offset

For guidance for lateral offset, see **Section 4.10.8**.

7.1.16 Structures

Bridges, culverts, walls, tunnels, and other structures must be designed in accordance with the current TxDOT **Bridge Design Manual – LFRD.** The design loading must be HL-93 design live load unless design for a special vehicle is specified or warranted.

Refer to the latest **TxDOT Bridge Design Manual - LFRD** for additional design guidance for bridges on TxDOT facilities.

7.1.17 Vertical Clearance

For vertical clearance requirements see **Section 4.8.6**.

7.1.18 Grade Separations, Interchanges, and Ramps

For guidance for grade separations, interchanges, and ramps, see **Chapter 15**.

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional information on grade separations, interchanges, and ramps.

7.1.19 Bicycle and Pedestrian Facilities

For guidance for Bicycle and Pedestrian Facilities, see **Chapters 18** and **19**. All projects should be assessed for pedestrian and bicycle accommodations that may be required.

7.1.20 Access Control

The installation of driveways and all access points along arterial highways must be in accordance with TxDOT's **AMM**.

For arterial highways constructed in developed or developing areas, it may be desirable to control access through right-of-way acquisition or by design (i.e., provision of frontage roads). Designed access control may be provided solely within 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.

On two-lane arterial highways, frontage roads or parallel service roads to serve small rural business communities or other developments should not be permitted. 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.

All frontage road development must be in accordance with the rules contained in **43 Texas Administrative Code (TAC) §15.54**. See **Section 8.1.21.4** for additional information on frontage road access control.

7.2 Rural Design Elements

7.2.1 General Characteristics

Arterials in rural areas range from two-lane roadways to multilane divided controlled access facilities. Rural arterial design criteria are used to facilitate high-speed and longer-distance travel between connections.

A smooth flowing alignment is desirable for a rural arterial and changes in vertical and horizontal alignment should be gradual. Minimum radii should be used sparingly to reduce crash rates and create a comfortable drive for travelers. Rural areas offer fewer physical constraints, allowing profiles to be generally flat and level with the surrounding ground terrain.

7.2.2 Typical Section

Rural arterial example typical sections are shown in **Figure 7-1** through **Figure 7-6** below. The geometric dimensions shown in these sections may be adjusted in accordance with **Section 7.1**.

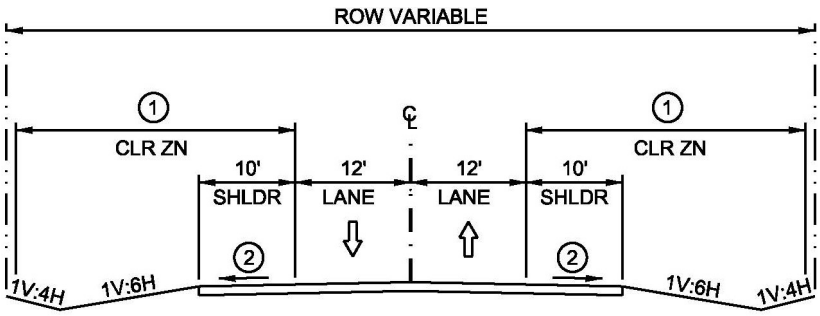
Selection of the appropriate cross-section is important for optimal safety and operational performance. **Table 7-2** provides guidelines on selecting optimized cross-sections for rural highways that are **not divided with a median barrier**. For additional guidance on roadway cross-section optimization, see **Section 7.2.5**.

Table 7-2: Potential Guidelines for Selecting Rural Cross Sections

AADT	Driveway Activity Index[1] per Mile	Truck Percentage	Preferred Cross-Section
≤ 3,000	Any	Any	Two-Lane Undivided / Two Lanes with TWLTL
3,000 – 15,000	≤ 30	Any	Super 2
	> 30	≤ 15%	Super 2 with TWLTL
		16% - 25%	Super 2 with TWLTL
		> 25%	Four Lanes with TWLTL
15,000 – 20,000	≤ 30	≤ 15%	Four Lanes with 4-ft Median Buffer[2]
		16% - 25%	Four Lanes with 4-ft Median Buffer[2]
		> 25%	Four Lanes with TWLTL
	> 30	≤ 15%	Four Lanes with 4-ft Median Buffer[2]
		16% - 25%	Four Lanes with TWLTL
		> 25%	Four Lanes with TWLTL
≥ 20,000	Any	Any	Four Lanes with TWLTL / Four-Lane Divided

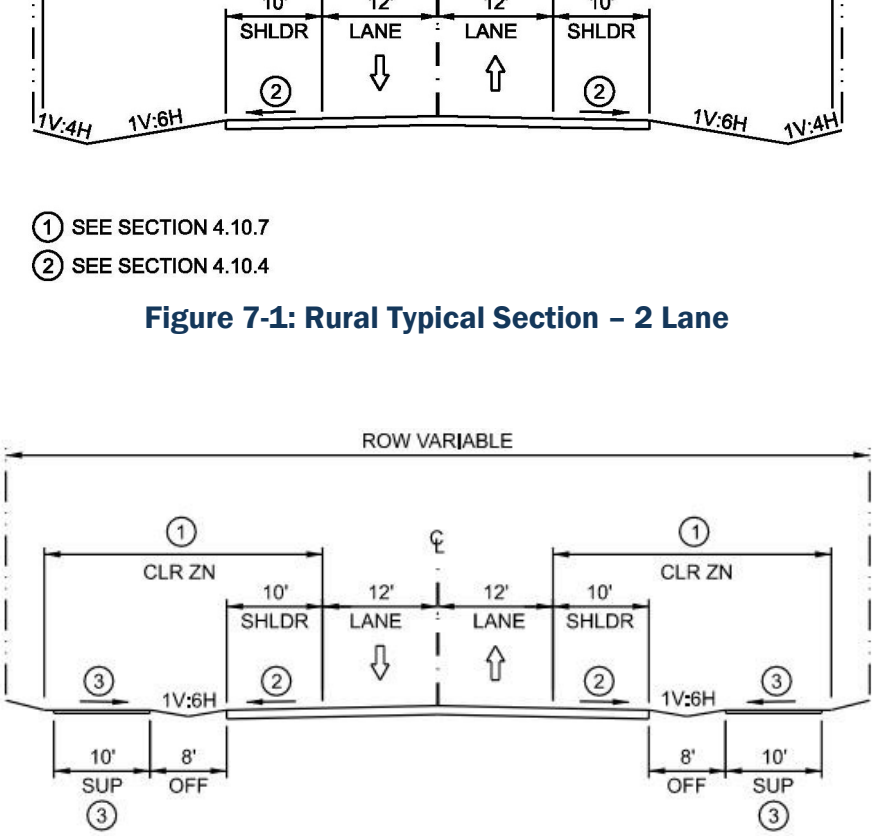
Notes:

1. Driveway activity index is the number of residential driveways. The index is equal to three times the number of industrial driveways, or 12 times the number of commercial driveways (measured per mile).
2. 6-ft minimum shoulder width. Greater shoulder widths are desirable.



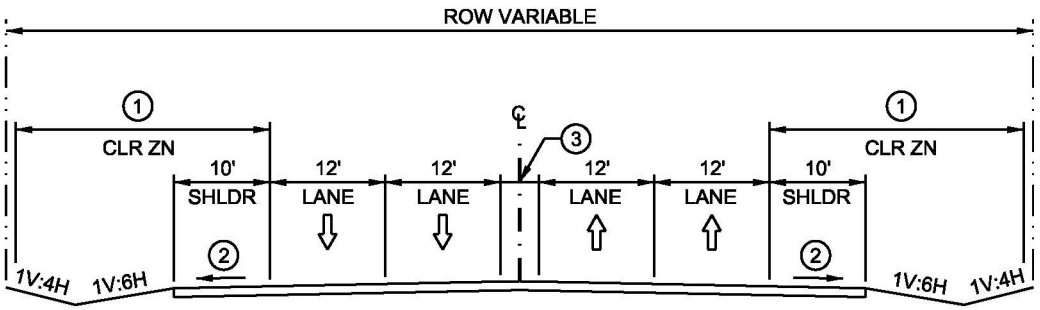
1 SEE SECTION 4.10.7
2 SEE SECTION 4.10.4

Figure 7-1: Rural Typical Section – 2 Lane



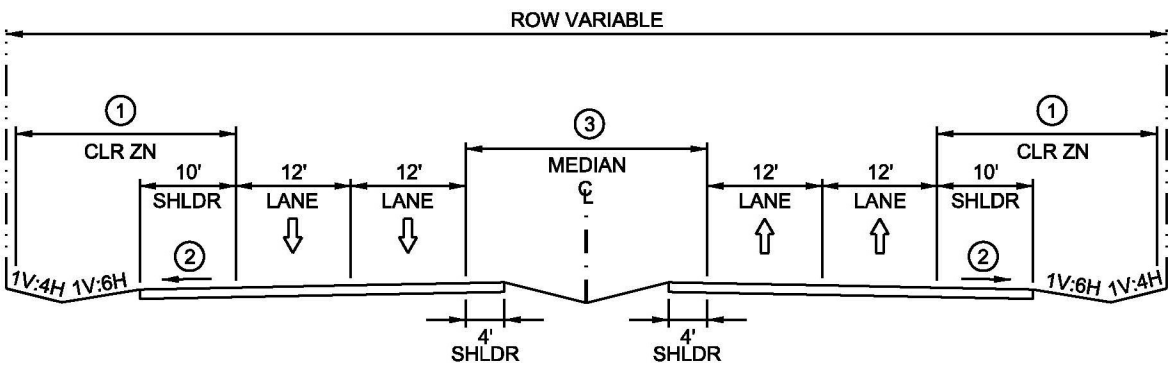
1 SEE SECTION 4.10.7
2 SEE SECTION 4.10.4
3 SEE CHAPTER 18

Figure 7-2: Rural Typical Section – 2 Lanes with SUP



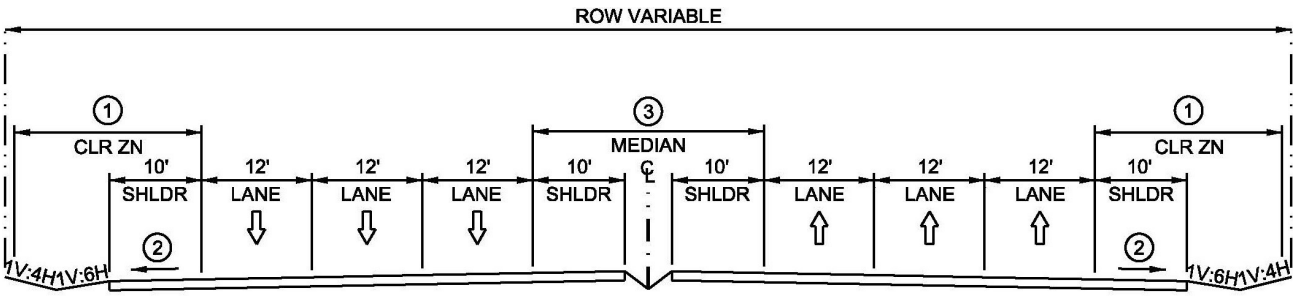
1 SEE SECTION 4.10.7
2 SEE SECTION 4.10.4
3 4' BUFFER PREFERRED

Figure 7-3: Rural Typical Section – Multilane Undivided with Median Buffer



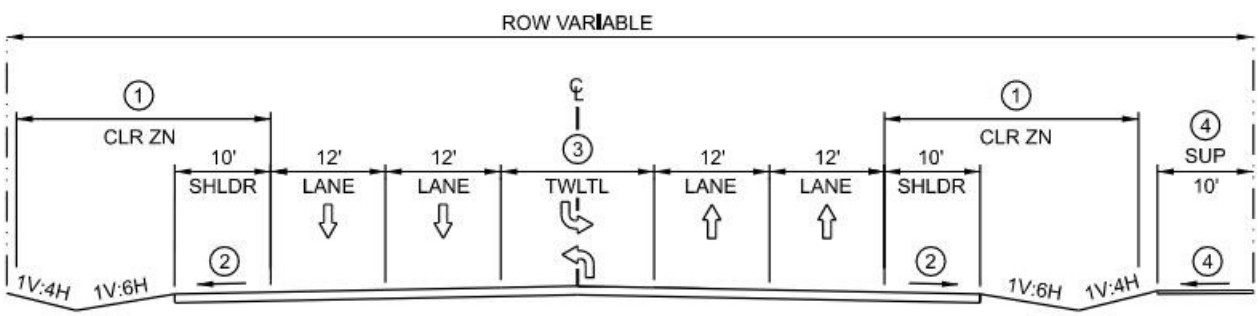
1 SEE SECTION 4.10.7
2 SEE SECTION 4.10.4
3 SEE SECTION 4.10.15

Figure 7-4: Rural Typical Section– Multilane Divided (4 Lanes)



(1) SEE SECTION 4.10.7
(2) SEE SECTION 4.10.4
(3) SEE SECTION 4.10.15

Figure 7-5: Rural Typical Section – Multilane Divided (6 or More Lanes)



(1) SEE SECTION 4.10.7
(2) SEE SECTION 4.10.4
(3) SEE SECTION 4.10.15.4
(4) SEE CHAPTER 18

Figure 7-6: Rural Typical Section – Multilane Undivided with TWLTL

7.2.3 Transitions to Four-Lane Divided Highways

Typical transitions from a two-lane to a four-lane divided highway are shown in **Figure 7-7**. **Transition geometrics 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.

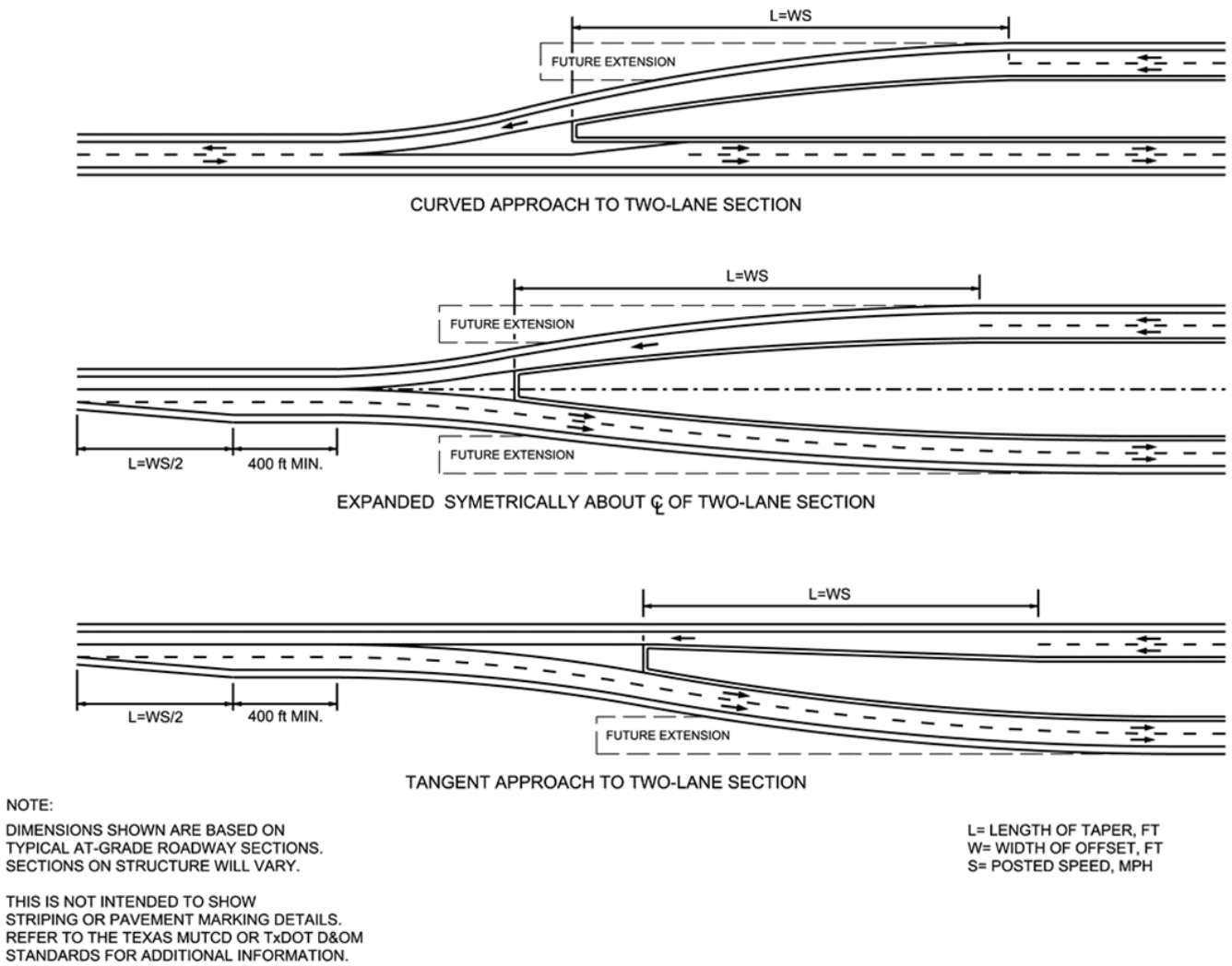


Figure 7-7: Typical Transistions from Two-Lane to Four-Lane Divided Highways

7.2.4 Converting 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 described in **Chapter 10**.

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 3R requirements for multi-lane facilities.

Transitions for opening and closing passing lanes on Super 2 highways is discussed in **Section 10.6**. Transitions for speed change lanes are discussed in **Section 4.10.2**.

7.2.5 Converting Two-Lane Roadways to Optimized Roadway Cross-sections

Four-lane undivided cross-sections have reduced safety and operational performance compared to other alternatives and should be avoided. **Table 7-2** lists preferred optimized cross-sections for various combinations of volume, driveway activity, and truck percentage. 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 that affect the feasibility of widening to a four-lane divided highway, conversion to a Super 2 highway, a two-lane highway with TWLTL, or a Super 2 with TWLTL may be considered to improve passing opportunities and traffic operations. **Figure 7-3**, **Figure 7-4**, and **Figure 7-6** include the general geometric typical section features for four-lane highways.

Before converting the existing two-lane roadway, a crash analysis should be conducted to identify specific areas with high crash frequencies so that corrective measures can be taken where appropriate. See **Chapter 10** for additional information on roadway cross-section optimization and Super 2 design.

7.3 Rural Town Design Elements

7.3.1 General Characteristics

The rural town context applies to roads in rural areas located within a developed community. Design of rural town arterials combines that of rural and urban arterial criteria that includes land use and pedestrian access along a specific corridor where both classifications are being utilized in a low population density area.

Arterials in rural town contexts can still carry high traffic densities and may have any number of through-traffic lanes along with varying lane and shoulder widths to meet LOS requirements. Rural town arterials have more constraints that include but are not limited to:

- Constrained ROW;
- Utilities;
- Surrounding buildings; and
- Local road connections.

Rural town arterials may include residential neighborhoods, schools, industrial facilities, main street business districts, each of which present differing design challenges and differing levels of pedestrian and bicycle activity.

7.3.2 Typical Section

Rural town arterial example typical sections are shown in **Figure 7-8** and **Figure 7-9** below. The geometric dimensions shown in these sections represent examples of different conditions and may be adjusted in accordance with **Section 7.1**.

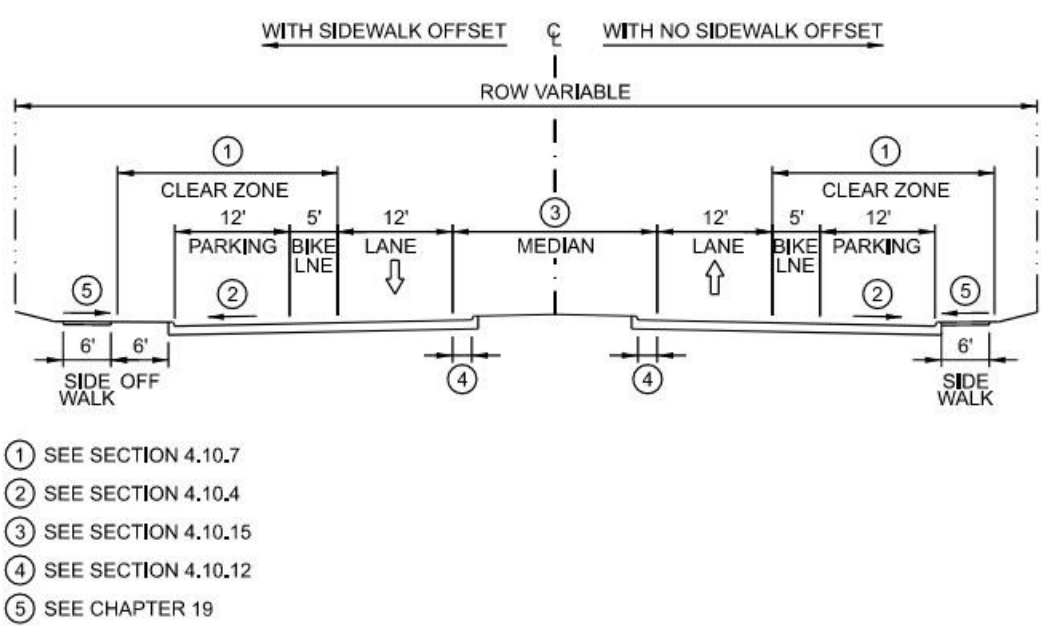


Figure 7-8: Rural Town Typical Section – Raised Median with Parking Lane

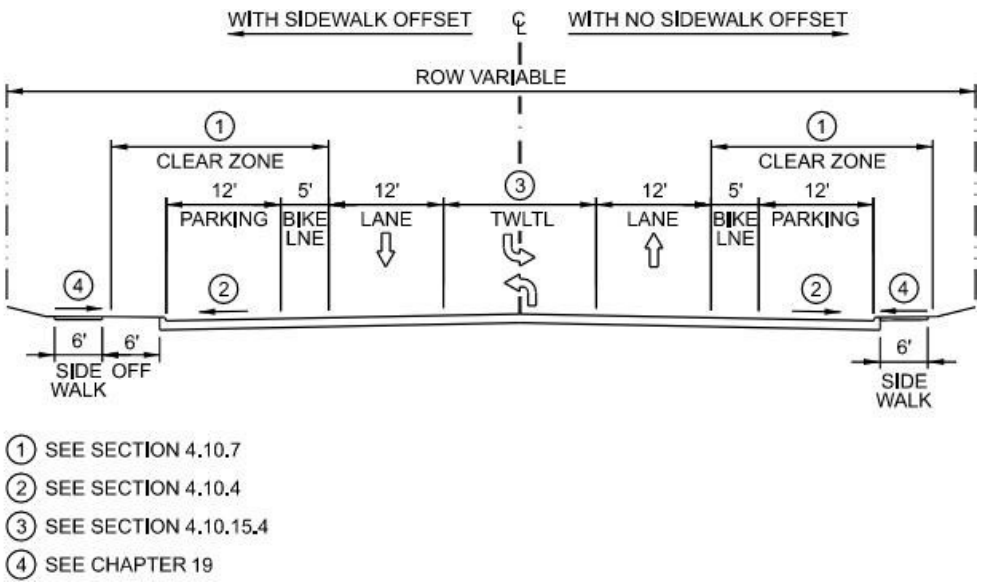


Figure 7-9: Rural Town Typical Section – Two-Way Left Turn Lane (TWLTL) with Parking Lane

7.4 Suburban Design Elements

7.4.1 General Characteristics

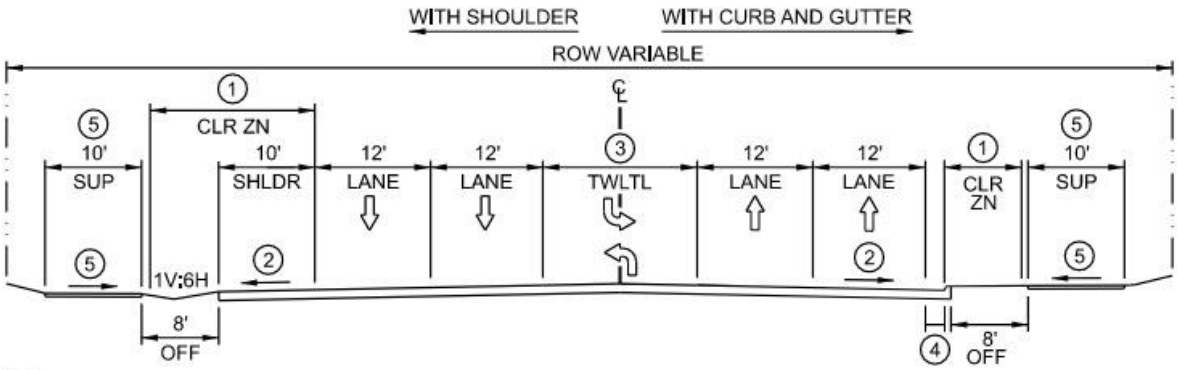
Arterials in suburban areas are similar to rural arterials in that they generally have higher operating speeds and carry multiple through travel lanes. However, due to the proximity of urban and rural town arterials, they may also have characteristics of these contexts as well. For example, these types of arterials may maintain high speeds (a rural characteristic) while utilizing curb and gutter to facilitate drainage (an urban and rural town characteristic).

Special care and attention must be given to transitioning the operating speed from high to low. This should be communicated to drivers by more than just speed limit sign changes. **Design features that gradually change from the rural cross section to rural town or urban cross section should be used to provide visual cues and positive guidance**. This may include gradual elimination or narrowing of the shoulder, narrowing of the travel lanes, use of curb and gutter, and center raised medians.

7.4.2 Typical Section

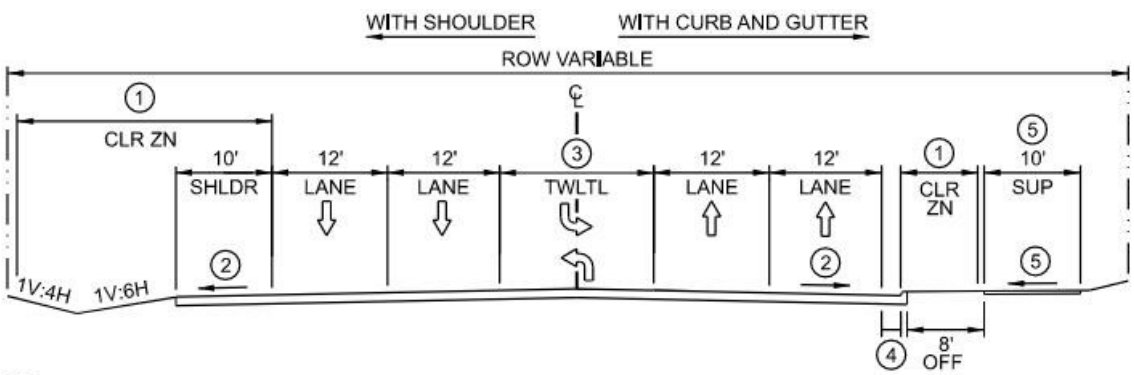
Suburban arterial example typical sections are shown in **Figure 7-10** through **Figure 7-13** below. The geometric dimensions shown in these sections may be adjusted in accordance with **Section 7.1**.

For suburban roadways with more urban characteristics, **Section 7.1.5.1** provides guidance on selecting the number of through lanes based on volume and refers to the **HCM** for more details. For suburban roadways with more rural characteristics, **Table 7-2** lists the preferred cross-section for various combinations of volume, driveway activity, and truck percentage.



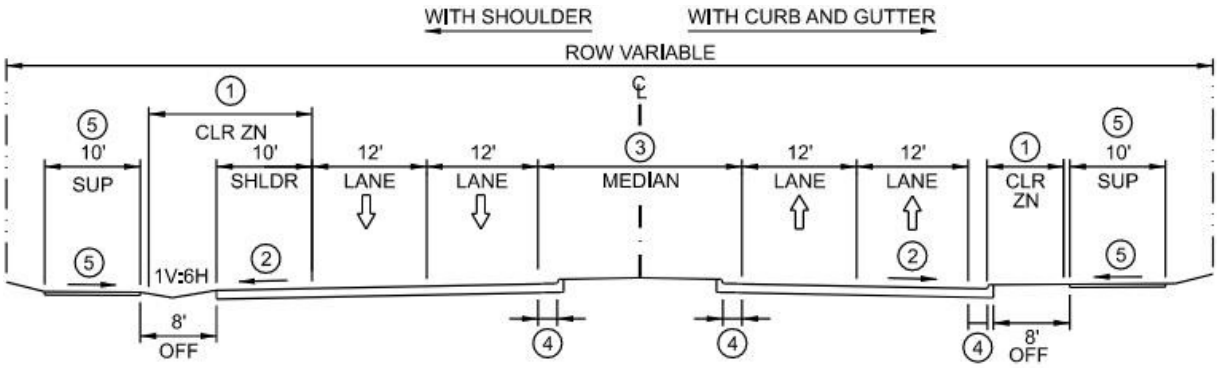
1. SEE SECTION 4.10.7
2. SEE SECTION 4.10.4
3. SEE SECTION 4.10.15.4
4. SEE SECTION 4.10.12
5. SEE CHAPTER 18

Figure 7-10: Suburban Typical Section – Two-Way Left Turn Lane (TWLTL)



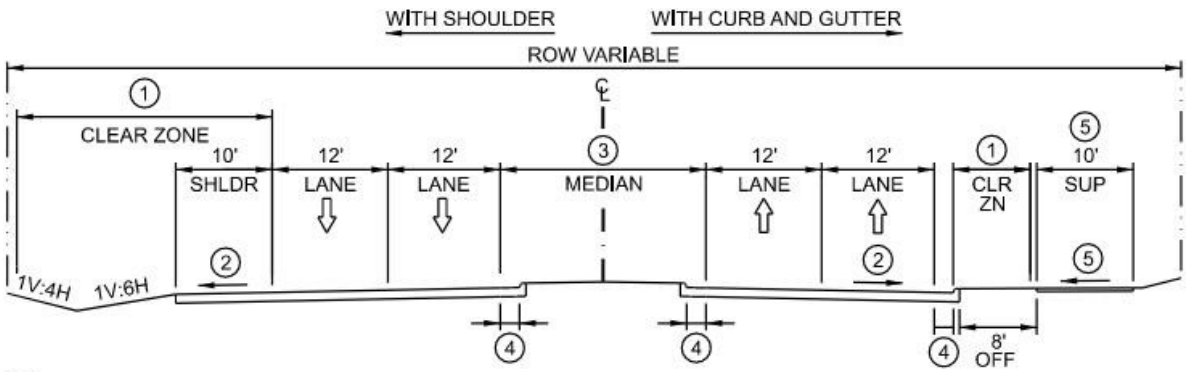
1. SEE SECTION 4.10.7
2. SEE SECTION 4.10.4
3. SEE SECTION 4.10.15.4
4. SEE SECTION 4.10.12
5. SEE CHAPTER 18

Figure 7-11: Suburban Typical Section -Two-Way Left Turn Lane (TWLTL) with Constrained ROW



1. SEE SECTION 4.10.7
2. SEE SECTION 4.10.4
3. SEE SECTION 4.10.15
4. SEE SECTION 4.10.12
5. SEE CHAPTER 18

Figure 7-12: Suburban Typical Section – Raised Median



1. SEE SECTION 4.10.7
2. SEE SECTION 4.10.4
3. SEE SECTION 4.10.15
4. SEE SECTION 4.10.12
5. SEE CHAPTER 18

Figure 7-13: Suburban Typical Section – Raised Median with Constrained ROW

7.5 Urban and Urban Core Design Elements

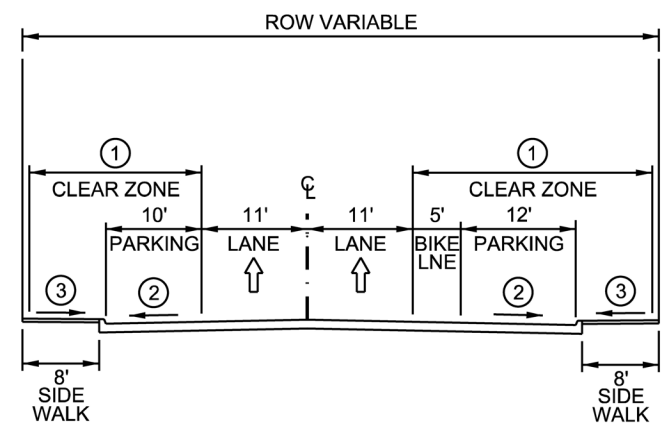
7.5.1 General Characteristics

Arterials in urban areas carry large or moderate traffic volumes with their design varying from fully controlled access freeways to two-lane streets.

A principal objective for an arterial in an urban area should be mobility of all users in an appropriate balance for the context of the facility and the appropriate degree of service to local development. The type of arterial selected is closely related to the level and quality of service desired for all users and to the context in which it is located.

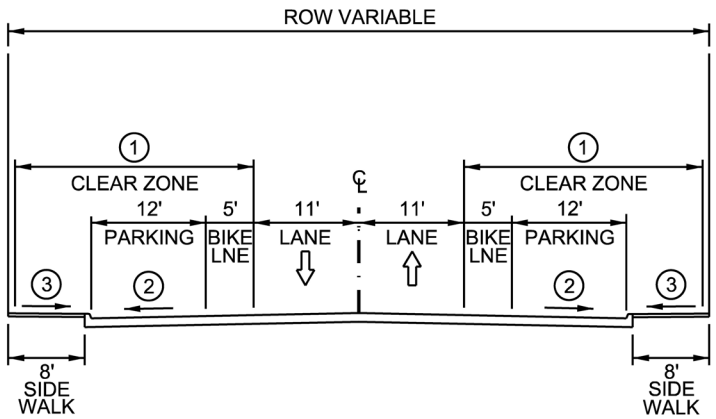
7.5.2 Typical Section

Urban and urban core arterial example typical sections are shown in **Figure 7-14** through **Figure 7-17** below. The geometric dimensions shown in these sections may be adjusted in accordance with **Section 7.1**.



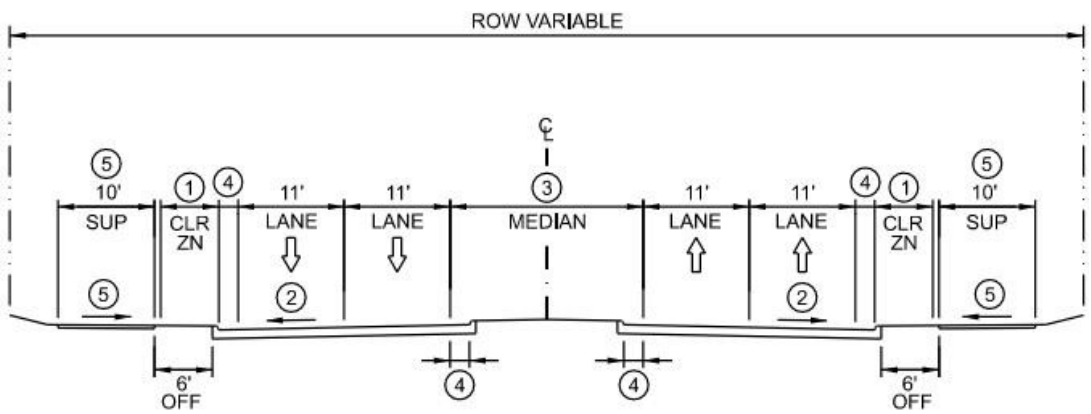
1 SEE SECTION 4.10.7
2 SEE SECTION 4.10.4
3 SEE CHAPTER 19

Figure 7-14: Urban Core Typical Section (One-Way)



1. SEE SECTION 4.10.7
2. SEE SECTION 4.10.4
3. SEE CHAPTER 19

Figure 7-15: Urban Core Typical Section (Two-Way)



1. SEE SECTION 4.10.7
2. SEE SECTION 4.10.4
3. SEE SECTION 4.10.15
4. SEE SECTION 4.10.12
5. SEE CHAPTER 18

Figure 7-16: Urban Typical Section – Separated Shared Use Paths (SUP)

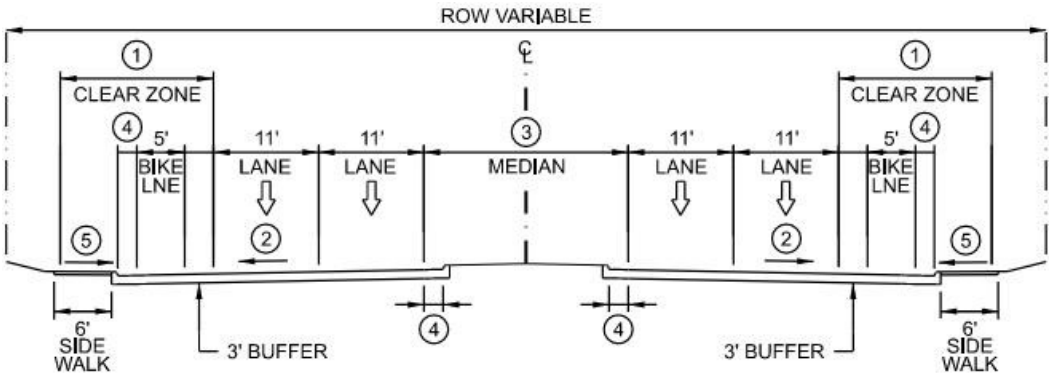


Figure 7-17: Urban Typical Section - Bike Lane

Chapter 8
Freeways (4R)

8.1 Design Considerations

8.1.1 Introduction

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.

Freeways may be provided in both rural and urban areas. All freeways in rural areas are designed for the rural context; the rural town context is generally not applicable to freeways.

Freeways in urban areas may be provided to fit all contexts including suburban, urban, and urban core.

Design criteria presented in this chapter is applicable to new construction of freeways, reconstruction of existing freeways and the conversion from a non-access controlled arterial to a freeway.

This chapter discusses the features and design criteria for freeways and includes the following sections and subsections:

8.1 Design Considerations

- 8.1.1 Introduction;
- 8.1.2 Target Design Values;
- 8.1.3 Design Speed;
- 8.1.4 Design Traffic Volumes;
- 8.1.5 Level of Service;
- 8.1.6 Travel Lane Width and Number of Lanes;
- 8.1.7 Shoulders;
- 8.1.8 Speed Change Lanes;
- 8.1.9 Medians;
- 8.1.10 Curbs;
- 8.1.11 Cross Slope and Superelevation;
- 8.1.12 Grades;
- 8.1.13 Roadside Design;
- 8.1.14 Structures;
- 8.1.15 Vertical Clearance;
- 8.1.16 Ramps and Terminals;
- 8.1.17 Outer Separations and Borders;
- 8.1.18 Frontage Roads;
- 8.1.19 Collector-Distributor Roads;
- 8.1.20 Crossing Facilities;
- 8.1.21 Access Control;
- 8.1.22 Managed Lanes and Toll Facilities; and
- 8.1.23 Tunnels.

8.2 Rural Design Elements

- 8.2.1 General Design Considerations

8.3 Suburban Design Elements

- 8.3.1 General Design Considerations

8.4 Urban Design Elements

- 8.4.1 General Design Considerations;
- 8.4.2 Depressed Freeways;
- 8.4.3 Elevated Freeways;
- 8.4.4 Ground Level Freeways;
- 8.4.5 Combination-Type Freeways; and
- 8.4.6 Special Freeway Designs.

8.5 Urban Core Design Elements

- 8.5.1 General Design Considerations

This chapter is organized with an introductory section on the general design considerations for freeways, followed by separate design discussions for freeways in urban, urban core, suburban, and rural contexts.

8.1.2 Target Design Values

Table 8-1 shows target design values of Freeways for each context classification. These design values may be adjusted based on the anticipated target speed and a performance-based practical design evaluation of the corridor as discussed in **Chapter 2**.

The subsections following **Table 8-1** provide additional detailed discussions for each design element and factors that may influence these values.

The design value thresholds for design exceptions and design waivers are presented in Appendix A and Appendix B respectively.

Table 8-1: Target Design Values for Freeway Mainlanes

		Design Element	Rural	Suburban	Urban	Urban Core	Reference
Roadway	★	Design Speed[1]	75 MPH	70 MPH	65 MPH	60 MPH	See 8.1.3
	★	Lane Width	12-ft				See 8.1.6
	★	Inside Shoulder Width (4-Lane Divided)	8-ft				See 8.1.7
	★	Inside Shoulder Width (6-Lane or More Divided)	12-ft				See 8.1.7
	★	Outside Shoulder Width	12-ft				See 8.1.7
		Speed Change Lane					
	⚑	- Lane Width	12-ft				See 8.1.8
	⚑	- Shoulder Width	10-ft				See 8.1.8
	⚑	Median Width	Varies				See 4.10.15 and 8.1.9
	★	Horizontal Curve Radius (Minimum)	See 4.7				
	★	Cross Slope on a Tangent (Typical)	2%				See 4.10.4
	★	Cross Slope on a Tangent (Maximum)	3%				See 4.10.4
	★	Superelevation Rate (Maximum)	See Table 4-3, Table 4-6, and Table 4-7				See 4.7.3

Design Element			Rural	Suburban	Urban	Urban Core	Reference
		Minimum Grade	0.25% (lined ditch channels), 0.3% (curbed facilities), or 0.5% (unpaved ditches)				See 4.8.1
	★	Maximum Grade (Level)	3%				See 4.8.1
	★	Maximum Grade (Rolling)	4%				See 4.8.1
	★	Vertical Clearance	18.5-ft				See 4.8.6, and 21.2
	★	Design Loading Structural Capacity	HL-93				See 8.1.14
	★	Stopping Sight Distance	See 4.11.1				
		Design Vehicle	WB-67				See 4.3.2
		Level of Service (LOS Design Year)	B	C	C	C	See 8.1.5
Roadside		Clear Zone	See Table 4-20				See 4.10.7
		Outer Separation	Varies				See 8.1.17
Notes	★	Denotes that this is a design exception condition if the specified values in Appendix A are not met.					
		Denotes that this is a design waiver condition if the specified values in Appendix B are not met.					
	1.	The selected Design Speed should meet the anticipated target speed of the facility during non-peak hours.					

8.1.3 Design Speed

The design speed of 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 (ROW) constraints, and socioeconomic costs. Minimum design speeds for freeways are shown in Appendix A.

See **Section 4.2.3** for additional guidance on selecting the appropriate design speed.

When determining applicable radii and superelevation for design speeds see Table 4-3 for a summary of superelevation methodologies to be used.

8.1.4 Design Traffic Volumes

Freeway projects in both rural and urban areas should be designed to accommodate traffic projections for a 20-year period into the future (mainlanes only). However, some elements (i.e., ramps, direct connections, frontage roads, and collector-distributor roads) may be based on shorter or longer design periods. For additional guidance on the selection of the appropriate periods for forecasting design traffic volumes, see **Section 4.3.1** or refer to **AASHTO's A Policy of Geometric Design of Highways and Streets**.

Segments of freeways may be constructed or reconstructed to be proportionate with intermediate traffic demands or with traffic

based on the completed system, whichever may be more appropriate.

8.1.5 Level of Service

For acceptable degrees of congestion, urban freeways, and their auxiliary facilities should generally be designed for Level of Service (LOS) C or better in the design year and can be referred to in the **HCM**. In heavily developed urban, or urban core contexts, LOS D may be acceptable. In heavily congested areas, other Measures 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, frontage roads, and collector-distributor roads) carrying unusually high volumes.

8.1.6 Travel Lane Width and Number of Lanes

The minimum and usual main lane 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 **HCM**. However, at a minimum, a freeway should have two through-traffic lanes in each direction of travel. See **Figure 8-3** for further information. **Figure 8-4** shows an alternative freeway section. The geometric dimensions shown in these figures represent examples of different conditions and may be adjusted in accordance with **Section 8.1.2**.

8.1.6.1 Principle of Lane Balance

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

- At entrance ramps, the number of lanes on the highway beyond the ramp should be equal to the total number of approach lanes on the highway and the lanes on the ramp, minus one. However, it can be equal to the total number of highway approach and ramp lanes;
- At exit ramps, the number of approach lanes on the highway should be equal to the sum of all lanes on the highway beyond the ramp and the lanes on the exit ramp, 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 entrance terminal taper 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 provided). 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;
- The traveled way of the highway should be reduced by not more than one traffic lane at a time;
- To satisfy lane-balance at two-lane exits, an auxiliary lane upstream of the exit should be provided; and

- 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 8-1**.

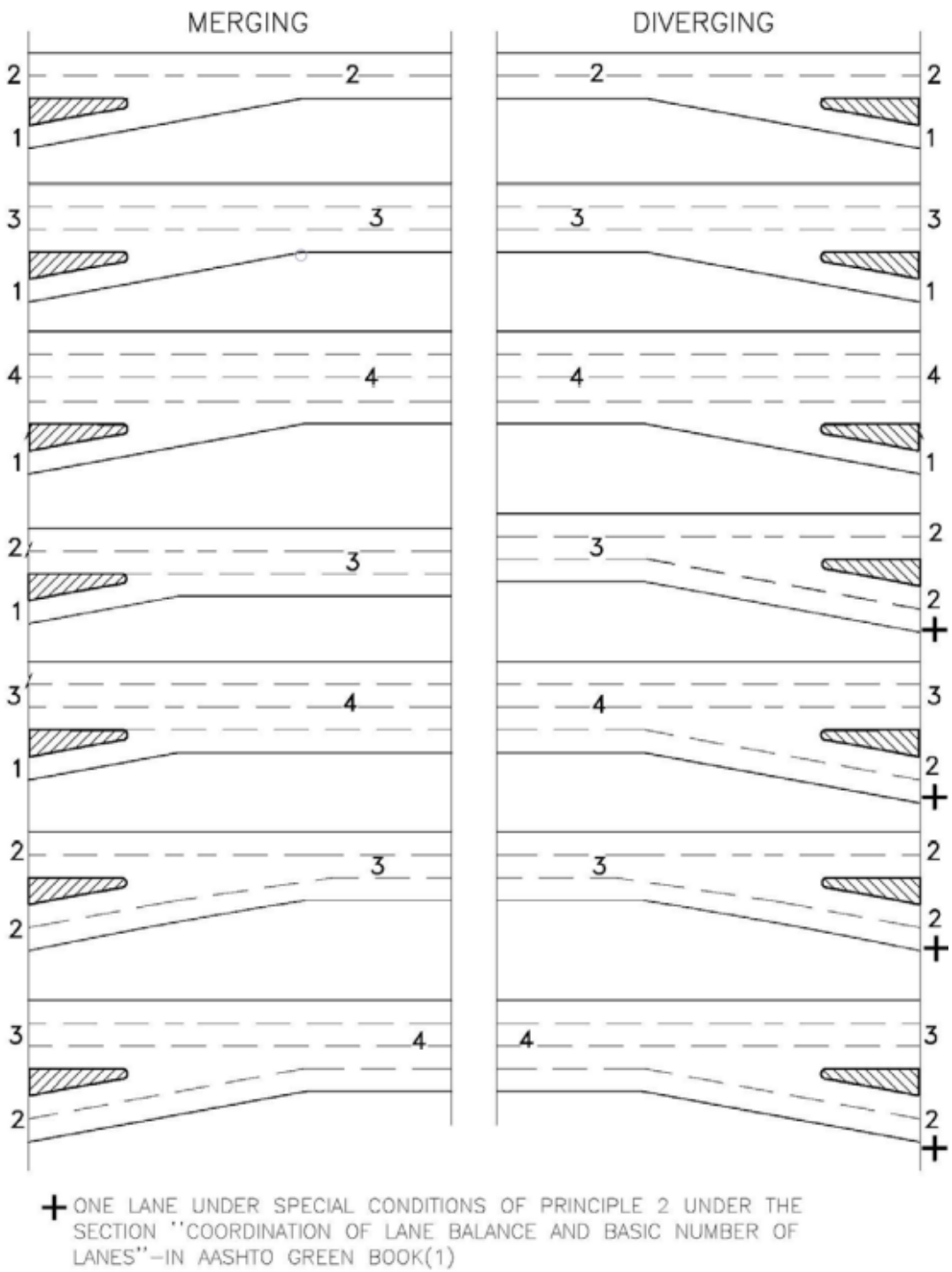
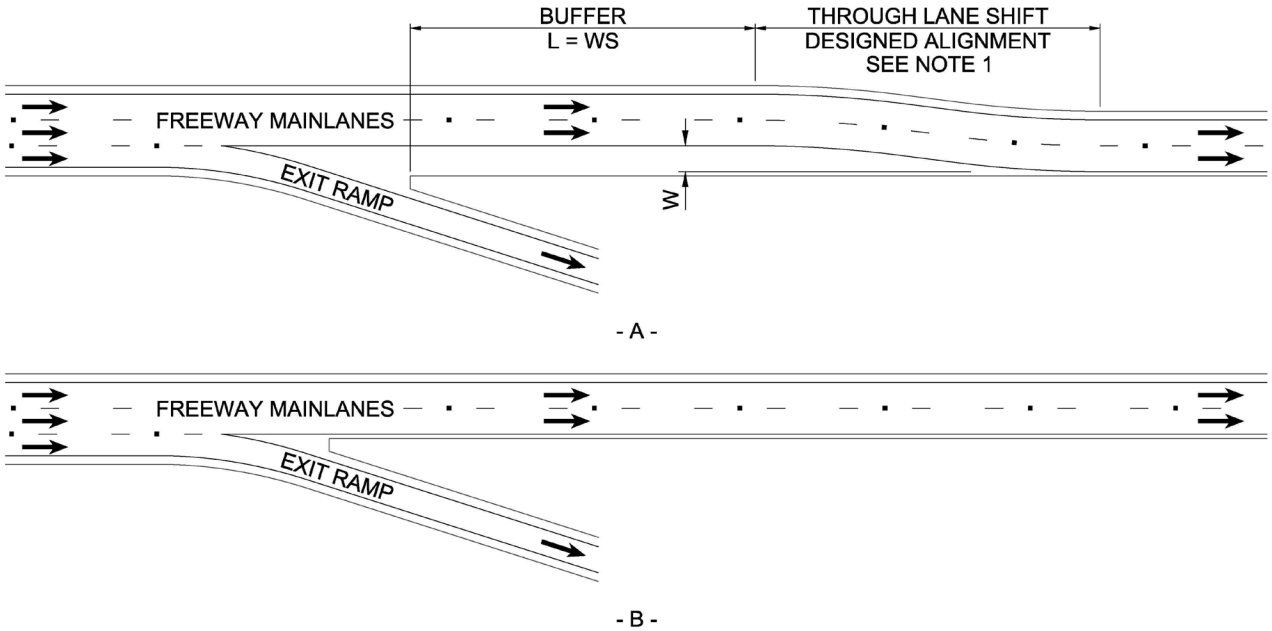


Figure 8-1 Typical Examples of Lane Balance

8.1.6.2 Left and Right Lane Drops

Sometimes it is necessary to reduce, or end, the number of through-travel lanes. When doing so, it is desirable to perform a lane drop at an exit ramp or a route split. When a lane reduction cannot take place at an exit ramp or route split, the lane reduction should occur in the outermost right through lane. **Left lane reductions should be avoided to minimize driver confusion**. When a left-through lane reduction is needed, the right lane should be dropped first, followed by a through lane shift or transition as shown in **Figure 8-2(A)**. The concepts shown in **Figure 8-2** is applicable to facilities that have 3 or more mainlanes. Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for further details.

Refer to the latest version of the **TMUTCD**, the **TxDOT Traffic Standards - Freeway Pavement Markings**, and TxDOT's **Freeway Signing Handbook** for additional guidance on signing and pavement marking design concerning lane drops.



NOTES:

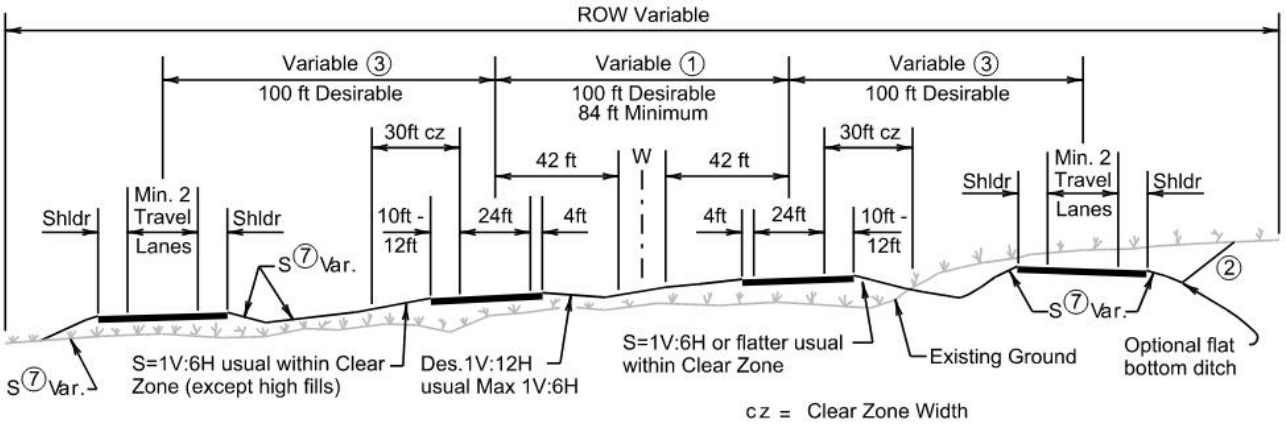
1. THE THROUGH LANE SHIFT SHOULD BE DESIGNED AS A HORIZONTAL ALIGNMENT IN ACCORDANCE WITH SECTION 4.7.

2. THIS IS NOT INTENDED TO SHOW STRIPING OR PAVEMENT MARKING DETAILS. REFER TO THE TEXAS MUTCD FOR ADDITIONAL INFORMATION.

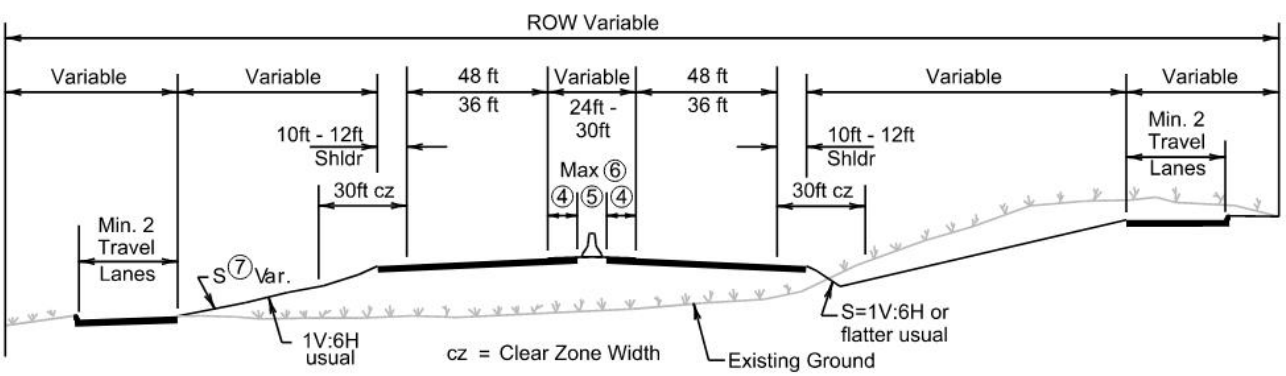
3. THIS FIGURE IS NOT INTENDED TO CONVEY LANE OR SHOULDER WIDTHS. REFER TO SECTION 8.1.7 AND 8.1.8 FOR LANE AND SHOULDER WIDTH INFORMATION.

4. L = LENGTH OF BUFFER
   W = WIDTH OF OFFSET FOR THROUGH LANE SHIFT
   S = DESIGN SPEED

Figure 8-2 Preferred Left Through Lane Reduction Example



TYPICAL RURAL FREEWAY SECTION (with Frontage Roads)



TYPICAL URBAN FREEWAY SECTION

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.

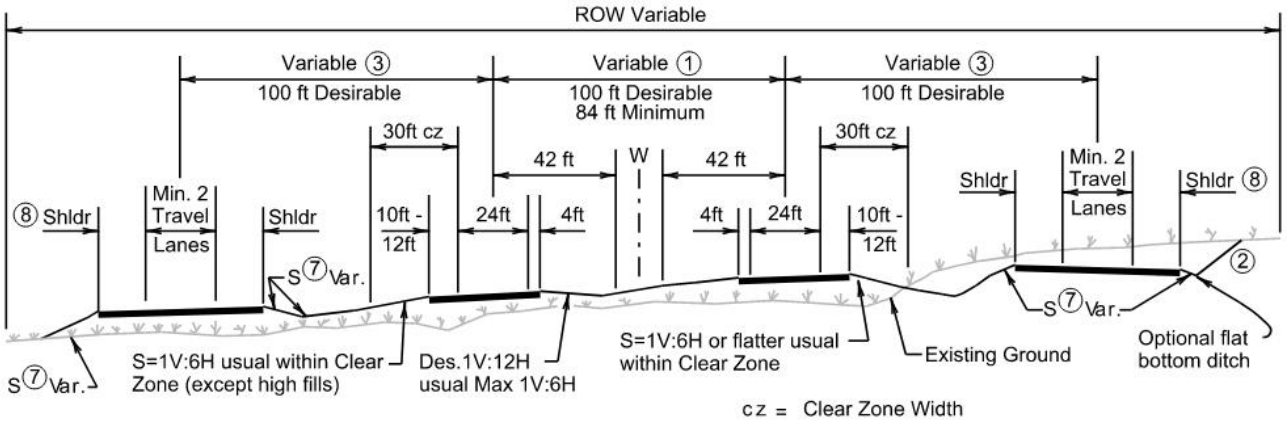
④ 10 ft - 12 ft on six lanes.

⑤ 4 ft typical. Median barrier generally used only in medians of 30 ft or less.

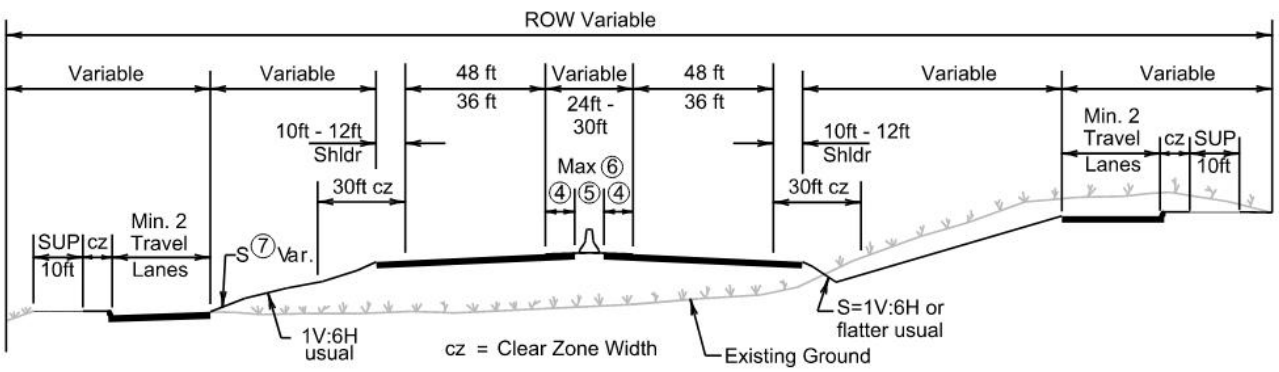
⑥ A 48 ft median (4-lane) or 52 ft (6-lane) median is appropriate where a future additional lane in each direction is planned.

⑦ See section 4.10 for discussion on slope rates.

Figure 8-3 Typical Freeway Sections



TYPICAL RURAL FREEWAY SECTION (with Frontage Roads and Bike Accessible Shoulder)



TYPICAL URBAN FREEWAY SECTION (with Shared Use Paths)

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 - 12 ft on six lanes.

⑤ 4 ft typical. Median barrier generally used only in medians of 30 ft or less.

⑥ A 48 ft median (4-lane) or 52 ft (6-lane) median is appropriate where a future additional lane in each direction is planned.

⑦ See section 4.10 for discussion on slope rates.

⑧ See Chapter 18 for discussion on Bike Accessible Shoulder.

Figure 8-4 Alternative Typical Freeway Sections

8.1.7 Shoulders

Freeways must provide continuous surfaced shoulders on each side of the mainlane roadways for all applicable contexts as shown in **Figure 8-3** and **Figure 8-4**.

For four-lane freeways, the minimum shoulder widths must be 10-ft on the outside and 4-ft on the inside (median side).

On freeways of six lanes or more, 10-ft inside shoulders must be provided for emergency pull-offs. 10-ft outside shoulder should be provided along all speed change lanes, and 12-ft outside shoulders are recommended when designated as emergency evacuation lanes.

8.1.8 Speed Change Lanes

A speed change lane or auxiliary lane is defined as the portion of roadway adjoining the through lanes for speed change, turning, storage for turning, weaving, truck climbing and other purposes that supplement through-traffic movement (e.g., maintaining appropriate lane balance between ramp terminals). Auxiliary lanes are typically provided on freeways to:

- Comply with the concept of lane balance;
- Comply with capacity needs;
- Accommodate speed changes;
- Accommodate weaving;
- Maneuvering of entering and exiting traffic; and
- Simplify traffic operations by reducing the number of lane changes.

The width of an auxiliary lane should be equal to or greater than the adjacent through lane.

When auxiliary lanes are provided along freeway mainlanes, the shoulder width should be 10-ft. However a 6-ft shoulder can be considered in weaving areas of LOS A or LOS B. A design waiver is not required for this condition.

The length of a weaving auxiliary lane should be determined through the procedures outlined in the **HCM** which is applicable to all freeway contexts. See **Section 15.6.3** or refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional guidance on the use of auxiliary lanes.

Figure 8-5 shows an example of a freeway auxiliary lane.

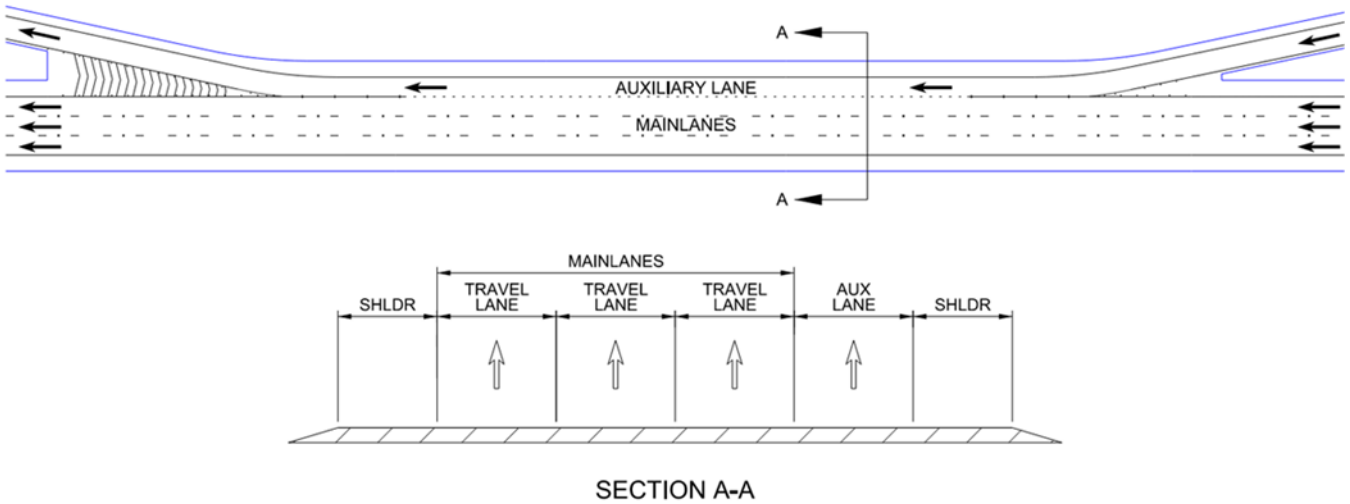


Figure 8-5 Example of Freeway Auxiliary Lane

8.1.9 Medians

The ultimate freeway section should be initially designed to reduce the adverse impact to high-speed and high-volume traffic resulting from construction in multiple phases when feasible. Several benefits can 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 a future need for additional lanes**. Provisions should be made for any future need for managed lanes such as 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. See **Section 4.10.15** for additional information on median design for freeways.

See **Section 17.9** for additional information on emergency freeway median crossings,

8.1.10 Curbs

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 must be traversable and should not be closer to the traveled way than the outer edge of the shoulder.

8.1.11 Cross Slope and Superelevation

For guidance on Cross Slope and Superelevation, see **Section 4.10.4** and **Section 4.7**.

8.1.12 Grades

Guidance for maximum Grades for freeways is presented in **Section 4.8.1.**

8.1.13 Roadside Design

Freeways must meet the minimum clear zone requirement shown in **Table 4-20**.

Freeways in urban areas often have restrictive rights-of-way which may require fixed objects (e.g., overhead sign supports, bridge piers, retaining walls, high mast lighting, etc.) to be placed within the clear zone. Fixed objects located within the clear zone must be protected or deemed crashworthy by the Department. Where fixed objects are located beyond the clear zone, roadway side slopes must be either traversable or protected. See **Section 4.10.11** for additional information on roadside design guidance.

8.1.14 Structures

Bridges, culverts, walls, tunnels, and other structures must be designed in accordance with the current **TxDOT Bridge Design Manual - LRFD.** The design loading must be HL-93 design live load unless design for a special vehicle is specified or warranted.

Structures that carry ramps must provide a clear width equal to the ramp width, including the ramp shoulders. The structure width and lateral clearance of highways and streets overpassing or underpassing the freeway are dependent on the functional class of the highway and street being underpassed or overpassed as discussed in **Chapters 5**, **6**, and **7**.

Refer to the latest **TxDOT Bridge Design Manual – LFRD** and **AASHTO's LRFD Bridge Design Specifications** for additional guidance on structures on TxDOT facilities.

8.1.15 Vertical Clearance

For guidance on Vertical Clearance, see **Section 4.8.6**.

8.1.16 Ramps and Terminals

For guidance on Ramps and Terminals for all freeways, see **Chapter 15.**

8.1.17 Outer Separations and Borders

The portion of the freeway between the mainlanes and frontage roads or ROW where frontage roads are not provided, should be wide enough to accommodate shoulders, auxiliary lanes, side slopes, drainage, retaining walls, ramps, signs, and other appurtenances necessary for traffic control.

Because of ROW limitations in urban areas, the outer separation may oftentimes be less than desired; however, in rural areas, where oncoming 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 **Section 4.10.7**, are preferable for errant vehicle recovery and safety.

The outer separation should be as wide as economically feasible to provide a buffer zone between the freeway and its adjacent area. Where practical, the outer separation should extend beyond the construction limits when frontage roads are not provided. **The widths of the outer separation will typically range from 80 to 150-ft in rural areas**. Urban areas will typically provide lesser widths, especially when retaining walls are used.

8.1.18 Frontage Roads

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

- Function and uses;
- Planning;
- Capacity and level of service;
- Frontage road design criteria; and
- Conversion of frontage roads from two-way to one-way operation.

8.1.18.1 Function and Uses

Frontage roads separate local traffic from the higher speed freeway traffic and provide access to properties along the freeway corridor. Frontage roads serve numerous functions, depending on the context of the freeway they serve and the features of the surrounding area.

They may be used to control access to the freeway, function as a facility serving adjacent properties, and maintain circulation of traffic on each side of the freeway. They can also provide invaluable operational flexibility, serving as detour routes during mainlane crashes, mainlane maintenance activity, inclement weather, and for over-height loads and buses. Continuous frontage roads provide the operational flexibility required to manage saturation on 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.

8.1.18.2 Planning

Frontage roads may be incorporated into a project at various points during 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. **A Frontage Road Briefing Document (FRBD)** must be developed, providing justification, and is approved by the District and then is submitted to Design Division prior to implementation of a new location frontage road (where no frontage road currently exists). An FRBD is also required when:

- Converting an existing two-way frontage road to a one-way frontage road;
- There is major reconstruction or rehabilitation of existing two-way frontage roads and two-way operation is to remain (requires approval from Design Division); or
- A frontage road is being proposed to become two-way (any type of project).

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 TAC §15.54**. Refer to the **PDP** for additional information.

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

8.1.18.3 Capacity and Level of Service

Although techniques to estimate capacity and level of service on freeways and urban arterials are detailed in the **HCM**, 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. **Research Report 1393-4F Procedures to Determine Frontage Road Level of Service and Ramp Spacing**. 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 (LOS) on a continuous frontage road section;
- Analyzing frontage road weaving sections; and
- Determining spacing requirements for ramp junctions.

8.1.18.4 Frontage Road Design Criteria

Frontage roads are classified as collectors and should be designed to match the values for a collector street or highway. See **Chapter 6** for additional information.

Any new frontage road **must be designed and constructed for one-way operation** unless prior approval from Design Division is obtained. There may be exceptions in certain isolated locations, where a one-way pattern would impose severe restrictions on circulation within an area. These exceptions must be approved by the Design Division during schematic design.

8.1.18.5 Conversions 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 and driveways;
- Increased potential for wrong-way entry to the mainlanes;
- Complicated intersections requiring turning movements from/to the Exit Ramps onto/from the frontage road, crossing the opposite directions of travel;

- Increased signal phasing and sequencing requirements as compared to those 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 routinely backs up from an intersection to within 100-ft of a freeway entrance or exit ramp gore;
- The LOS of a signalized intersection on the frontage road drops 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 intersection;
- 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 crossover interchanges spaced more than two miles apart, require consideration of additional crossover interchanges to minimize the distance traveled for adjacent residents and business patrons. Existing local street systems in the area should facilitate traffic circulation and minimize the travel time impact of converting frontage roads from two-way to one-way operation.

The conversion of two-way to one-way frontage roads should be accomplished with ramp and terminal design based on reconstruction criteria shown in **Chapter 15**. For existing frontage road lanes may retain dimensions that meet minimum criteria, see **Appendix A** and **Appendix B.**

Two-way frontage roads should only be considered in rural areas where:

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

8.1.19 Collector-Distributor Roads

A collector-distributor (C-D) road is a road that parallels and connects the main travel lanes and frontage roads or entrance ramps. The purpose of a C-D road is to minimize weaving on the main travel lanes of a freeway by reducing the number of entrance and exit points on the through lanes while satisfying the demand for access to and from the freeway.

C-D roads are limited access roadways provided within a single interchange or continuously through two or more interchanges along a freeway corridor. They are similar to frontage roads except that access to abutting properties is not permitted.

The minimum lane arrangement for a C-D system is two C-D lanes and two mainlane through lanes, as shown in **Figure 8-6**; however, other combinations may be developed as capacity needs warrant.

Continuous C-D roads should be integrated to the main travel lane design to develop an overall system. **C-D roads should have shoulders equal to the width of the main travel lane shoulders.**

Sometimes C-D roads are referred to as an "Intersection Bypass". See **Figure 8-7** for an example of this application. In this case, a C-D road is similar to an exit ramp but is typically longer than a ramp. A C-D 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
- Enhance safety by providing a place for exiting traffic to wait to access cross streets, rather than backing up onto high-speed mainlanes.

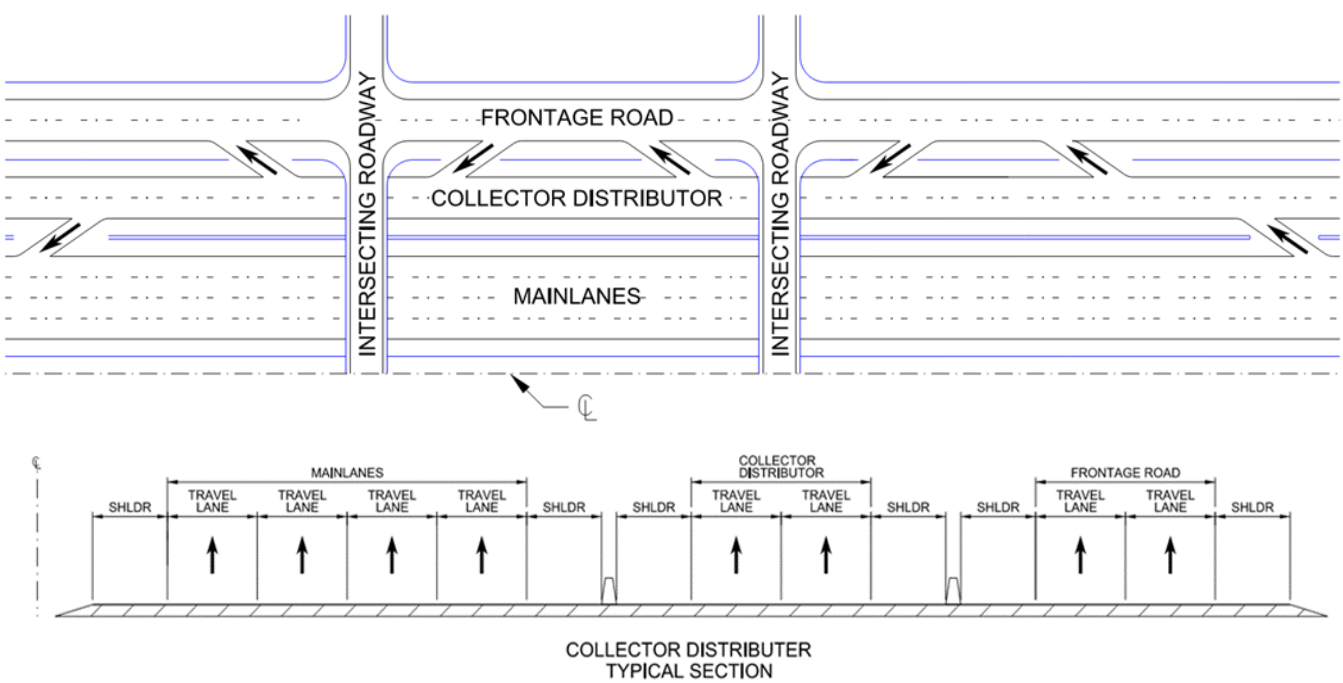


Figure 8-6 Example of a Collector-Distributor System

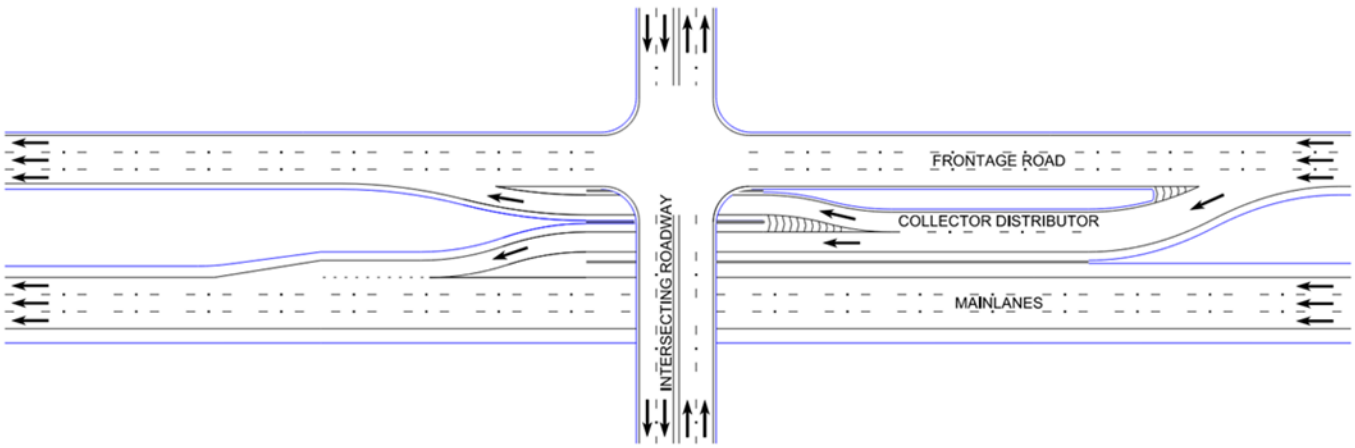


Figure 8-7 Example of an Intersection Bypass Collector-Distributor

C-D roads are a desirable 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. See **Figure 8-8** for an example of a cloverleaf interchange C-D road.

8.1.19.1 Collector-Distributor Design Criteria

The following design criteria should be used for the design of collector-distributor roads:

- Design Speed: No more than a 10 mph reduction from mainlane design speed;
- Lane Width: freeway mainlane width (**Figure 8-1**); and
- Capacity analysis and basic lane determination should be performed for the overall system rather than for the separate roadways.

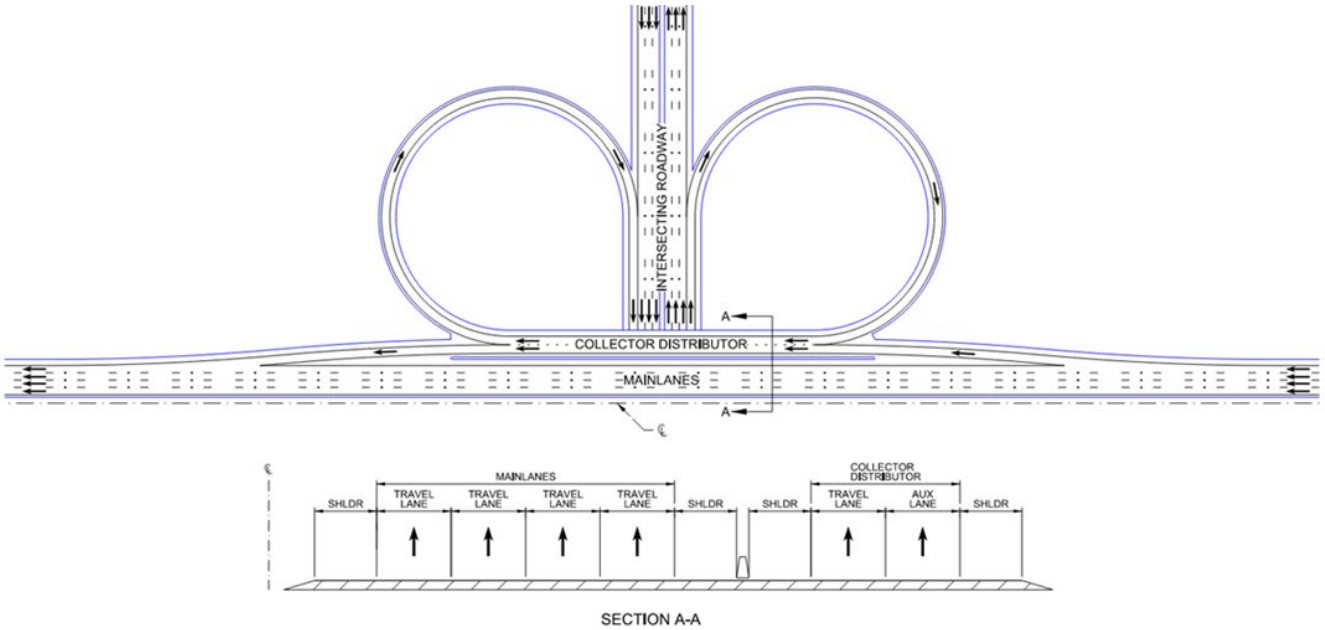


Figure 8-8 Example of a Cloverleaf Interchange Collector-Distributor

8.1.20 Crossing Facilities

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

- Local Roadways: **Section 5.1**;
- Collectors: **Section 6.1**;
- Arterials: **Section 7.1**; and

Freeways: **Table 8-1** and **Chapter 15**.

8.1.21 Access Control

This subsection discusses access control and includes the following topics:

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

8.1.21.1 General Guidance

The entire Interstate Highway System and portions of the State Highway System have been designated by the Texas Transportation Commission as Controlled Access Highways (refer to the **Statewide Planning Map**). 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 (e.g., driveway permitting), which is an inherent right of sovereignty, which may entitle the owner to potential damages suffered by the loss of such access.

Refer to TxDOT's **AMM** for additional information on the determination of control of access limits.

8.1.21.2 Control of Access Methods

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

- Designation (**TTC §203.031 Control of Access**); and
- Design (continuous frontage road and State's police power). Refer to TxDOT's **AMM** for additional information.

8.1.21.2.1 Control of Access by Designation

When the Texas Transportation Commission designates a freeway as a controlled access facility under **TTC §203.031**, the State is empowered to control access through access restrictions. All Interstate Highways are designated as controlled access facilities as well as other routes that have been or may be designated by Commission Minute Order. These designated freeways may or may not have frontage roads, whichever arrangement is determined to be appropriate during Project Scoping. Development of freeways by designation, rather than solely by design, is the preferred design approach especially for all new location freeways.

8.1.21.2.2 Control of Access by Design

TxDOT is not empowered to purchase access rights. If an existing highway is to be developed as a controlled access facility solely by design (i.e., not designated by the Texas Transportation Commission), 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. The State 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.

Refer to TxDOT's **AMM** for additional guidance on access requirements.

The design philosophy from the Frontage Roads Access section below 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 8-9** and **Figure 8-10**.

Whenever access is to be controlled solely by provision of frontage roads, State's 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 8-9** and **Figure 8-10** is preferred over controlling access solely by design (State's police power).

Some designated controlled access freeways may be a combination of new location and along an existing road. **Figure 8-9**, **Figure 8-10**, and State's Police Powers may be used at appropriate locations.

8.1.21.3 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.

8.1.21.4 Frontage Road Access

Information on the driveway clearances from the cross-street intersection is contained in TxDOT's **AMM** 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.

Frontage road access should be controlled by imposing recommended access restrictions in accordance with **Figure 8-9** and **Figure 8-10** whenever both of the following conditions prevail:

- ROW is being obtained from the abutting property owner(s); and
- A landlocked condition does not result;

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

- No ROW 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 practicable.

The placement of streets and driveways near freeway ramp junction with frontage road should be carefully considered and permitted only after local traffic operations are considered.

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 practical, then deviation from these recommended guidelines should be supported by a traffic operations and safety evaluation.

8.1.21.4.1 Access Beyond Exit Ramps

Figure 8-9 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 8-2** show the desirable spacing to be used

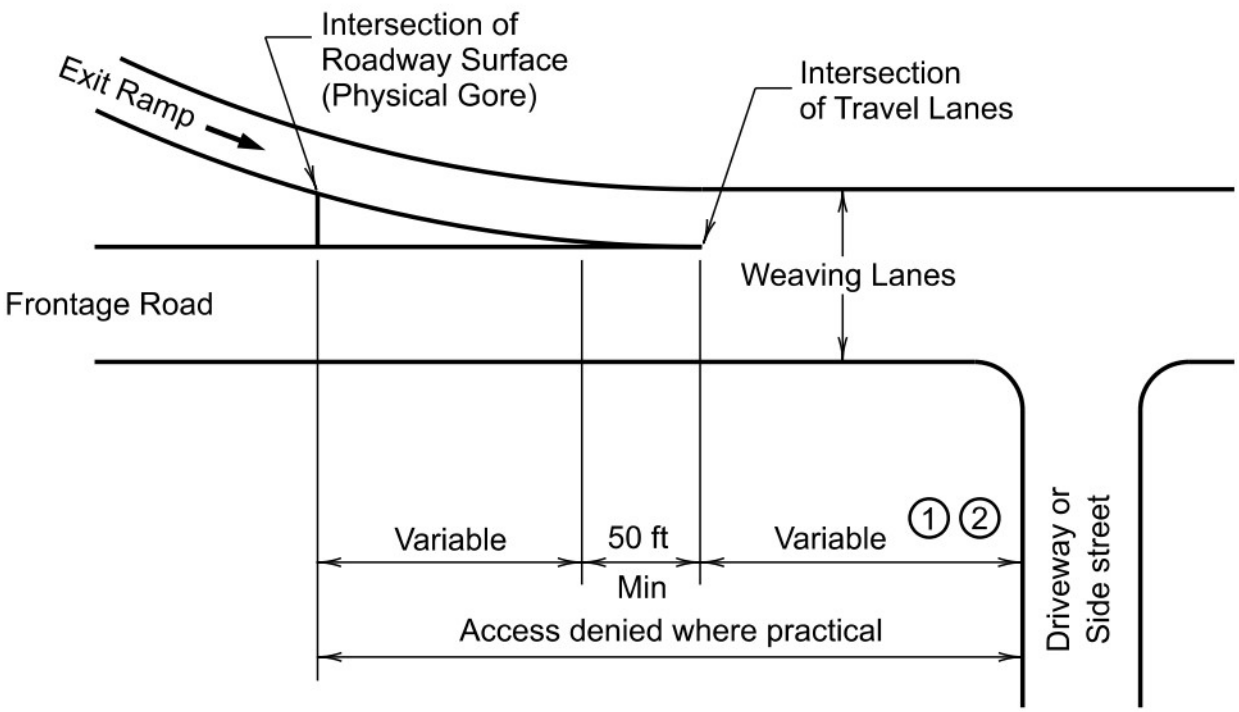
between exit ramps and driveways, side streets, or cross streets if practical.

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. It is desirable but not required to maintain separation between the Intersection of Travel Lanes (see **Figure 8-9** and **Figure 8-10**) and downstream driveways or side streets.

It is recognized that there are occasions when meeting the exit ramp separation distance values in **Table 8-2** 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, including curbed medians or other channelization that would restrict access to driveways within this 250-ft distance.

Refer to the **TMUTCD** for channelization options.



① For driveway, side street, or cross street to entrance ramp spacings, see desirable values in Table 8-2.

② When the spacing requirements in Table 8-2 cannot be met and there are no negative operational or safety impacts, at least 250-ft of separation should be provided from the intersection of the travel lane to the driveway or street.

NOTE: REFER TO THE TEXAS MUTCD FOR CHANNELIZATION AND PAVEMENT MARKING DETAILS.

Figure 8-9 Recommended Access Control at Exit Ramp Junction with Frontage Road

Table 8-2 Desirable Spacing between Exit Ramps and Driveways, Side Streets, or Cross Streets

Total Projected Frontage Road + Ramp Volume (vph)	Driveway or Side Street Projected 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
--	> 1000	1000	460	560
> 2500	< 250	920	460	560
--	> 250	950	460	560
--	>750	1000	600	690
--	>1000	1000	1000	1000

8.1.21.4.2 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 8-10 shows the recommended access control strategy for planned entrance ramps and should be used where practical. There will be occasions when meeting the entrance ramp separation distance values shown in **Figure 8-10** 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 **TMUTCD** for specific types of channelization.

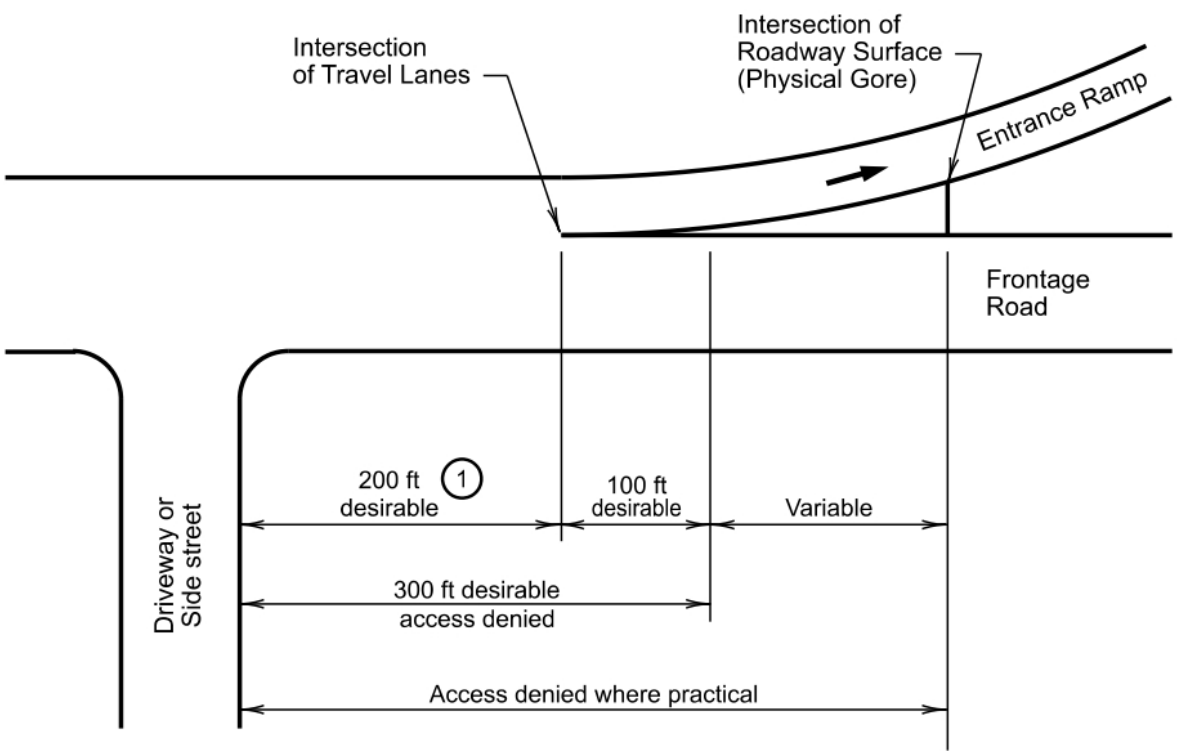
8.1.21.4.3 Ramp Location

During schematic development, care should be exercised to develop the design in sufficient detail to accurately tie down the locations of ramp junctions with frontage roads and the location of access control limits. Refer to TxDOT's **PDP** for specific guidance on schematic development.

These drawings are often displayed at meetings and hearings and become the basis for ROW instruments or the department's regulation of driveway location for that project.

Figure 8-9 and **Figure 8-10** provide recommended access control at exit and entrance ramp junctions with frontage roads.

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



1 When the desirable spacing cannot be met and there are no negative operational or safety impacts, at least 100 ft of separation should be provided from the intersection of the travel lane to the driveway or street.

NOTE: REFER TO THE TEXAS MUTCD FOR CHANNELIZATION AND PAVEMENT MARKING DETAILS.

Figure 8-10 Recommended Access Control at Entrance Ramp Junction with Frontage Road.

8.1.21.4.4 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; and
- **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 refer to TxDOT's **AMM**.

8.1.22 Managed Lanes and Toll Facilities

Managed lanes are defined by the FHWA as highway facilities or a set of lanes where operational strategies are proactively implemented and managed in response to changing conditions. This increases freeway efficiency, maximizes capacity, manages freeway demand, and generates revenue. Examples of managed lanes include high-occupancy vehicle (HOV) lanes, value-priced lanes, high occupancy toll (HOT) lanes, and exclusive or special use lanes such as express lanes, bus lanes, transit lanes, and reversible flow lanes.

The following geometric design elements and related topics should be considered in the design of a managed lane facility:

- Geometric and operational consistency;
- Design speed;
- Cross sectional elements and alignment;
- Separation to general-purpose lanes;
- Access; and
- Drainage.

Refer to **NCHRP Report 835 Guidelines for Implementing Managed Lanes** or **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional information on managed lane facilities.

A Design Exception would be required if the lane and shoulder widths shown in Figure 8-11 are not met.

8.1.22.1 Geometric and Operational Consistency

The geometric design of managed lane and general-purpose facilities (i.e., freeway mainlanes) should be based on design principles and criteria for freeways. This reinforces driver expectations and promotes system connectivity.

Managed lanes are typically constructed as dedicated lanes within a freeway facility or in some cases, a freeway within a freeway. See **Figure 8-11** for an example typical section.

Managed lane design should match the design elements and traffic control devices on the adjacent freeway facility so that there is an

established driver expectancy between both facilities. For example, if the adjacent freeway provides 50-mph ramps, a similar design should be applied to the ramps that connect the managed lanes and general-purpose lanes.

8.1.22.2 Design Speed

The design speed of a managed lane facility, including ramps and direct connectors, should match the design speed of the adjoining freeway facility.

This is especially true if there is a possibility of the managed lane facility being used by general-purpose traffic during the off-peak hours or at some time in the future. If there is no possibility that the facility will be used by general purpose traffic, and if use of this facility will be limited to a single vehicle type, such as buses, then the specific physical dimensions and operating characteristics of that vehicle should be considered in the design.

8.1.22.3 Cross-Sectional Elements and Alignment

8.1.22.3.1 Context Classification

Managed lane facilities must match the context classification of the adjoining freeway facility.

8.1.22.3.2 Number of Lanes

Planning studies should be performed in conformance with a **HCM** analysis to confirm the number of through lanes needed for the managed lane system. This is particularly important when considering system-level demand inputs and outputs for a regional network of managed lanes.

The level of service for the managed lane facility must provide a greater level of service than the adjacent freeway facility.

8.1.22.3.3 Width of Lanes

The recommended and minimum lane width for a managed lane facility is 12-ft.

8.1.22.3.4 Width of Shoulders

For desirable and minimum shoulder widths refer to **NCHRP Report 835** Chapter 3.

8.1.22.3.5 Median

Medians on managed lane facilities should be designed in the same manner as freeway medians. For guidance on freeway Medians, see **Section 8.1.9.**

8.1.22.3.6 Horizontal Alignment

For guidance on Horizontal Alignment, see **Section 4.7.**

8.1.22.3.7 Vertical Alignment

For guidance on Vertical Alignment design, see **Section 4.8**. Maximum grades for managed lane facilities must follow the requirements of freeways.

8.1.22.3.8 Cross Slope and Superelevation

For guidance on Cross Slope and Superelevation, see **Section 4.10.4** and **Section 4.7**.

8.1.22.3.9 Clear Zone

For Clear Zone requirements for managed lane facilities, see **Section 4.10.7** for freeways.

8.1.22.3.10 Vertical Clearance

For guidance on Vertical Clearance, see **Section 4.8.6**.

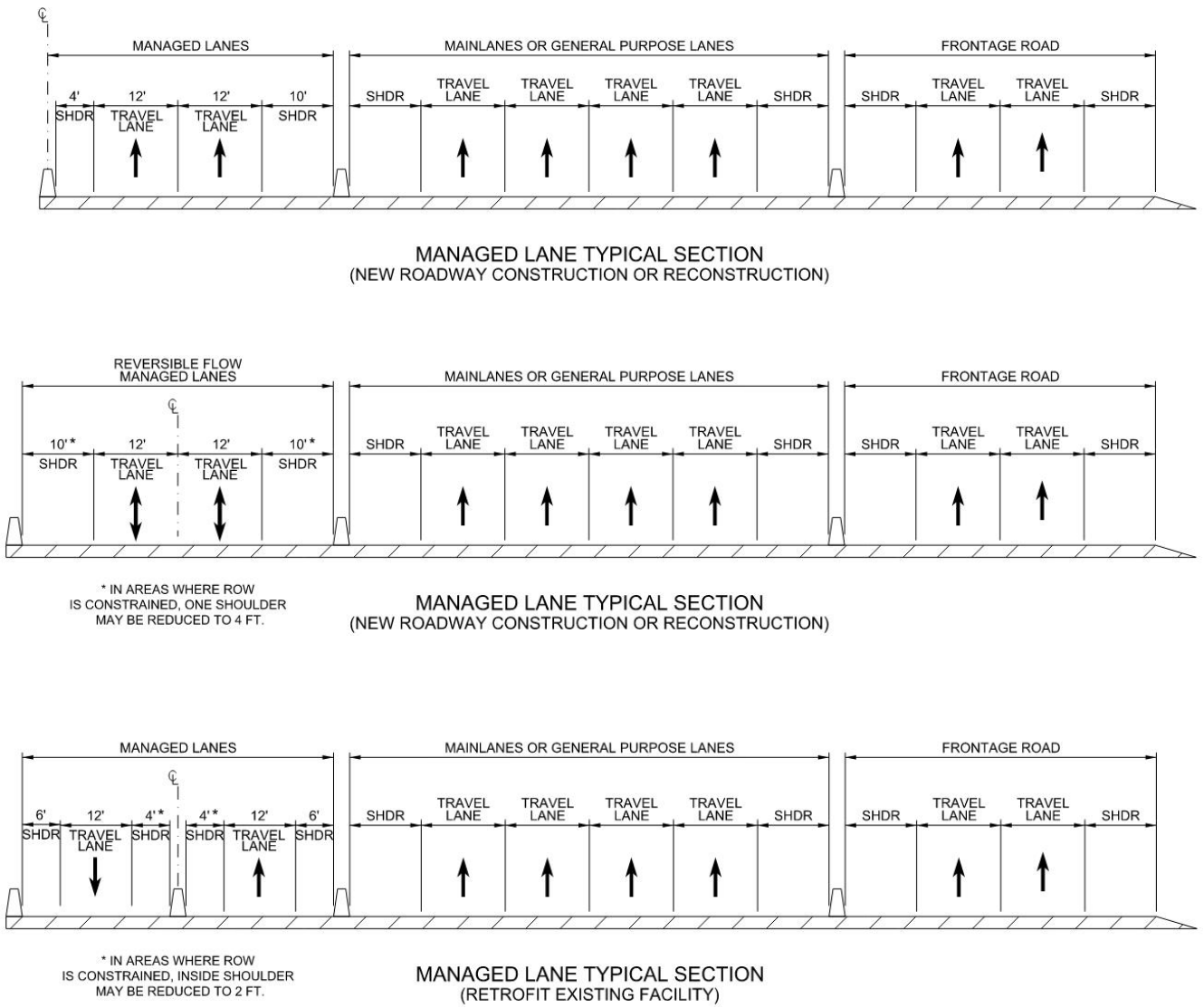


Figure 8-11 Example Managed Lane Typical Section

8.1.22.4 Separation to General Purpose Lanes

There are multiple ways to provide separation between managed lanes and the general-purpose lanes of a freeway. Barriers, buffers, flexible delineators, pavement markings, and separate facilities can all be used. In some cases, facilities may have a combination of separation types (e.g., a mixture of buffer separation and pavement marking separation where cross section is limited).

For additional information on separation methods for managed lane facilities, refer to **NCHRP Report 835 Guidelines for Implementing Managed Lanes**.

8.1.22.5 Access

Access to and from the managed lane facility must follow the ramp design guidance, see **Section 15.7**.

In situations where weaving is required on the managed lane facility, the minimum ramp spacing must be determined with the analysis procedures outlined in the **HCM**. For early planning purposes, the applicable weaving distances on a freeway between two system interchanges, a system interchange and service interchange, or two service interchanges should be used.

Access to managed lane facilities can either be accomplished through a continuous access or limited access approach. The selection of access type is based on a general evaluation of the performance and management benefits for the entire freeway system as well as the costs of constructing and operating the managed lanes.

8.1.22.6 Drainage

Drainage criteria for managed lanes is identical to the drainage criteria of freeways. Refer to TxDOT's **HDM** for drainage criteria requirements for freeways.

For managed lanes located to the left of general-purpose lanes, drainage is a particular consideration when the lanes are separated by a median barrier. In these scenarios, the shoulder width and inlet spacing must be designed to handle the required ponded width.

8.1.23 Tunnels

Development of highways and freeways may include sections constructed in tunnels to either carry the roadway under or through a natural obstacle or to minimize the effect of the roadway on the community. General conditions under which tunnel construction may be warranted include:

- Long, narrow terrain ridges where a cut section may either be more costly or carry environmental consequences;
- Narrow ROW where all of the surface area is needed for street purposes;
- Large intersection areas or a series of adjoining intersections on an irregular or diagonal street pattern;
- Grade-separated pedestrian and/or bicycle facilities are needed;
- Railroad yards, airport runways, or similar facilities;
- Parks or similar land uses (existing or planned); or
- Locations where ROW acquisition costs exceed cost of tunnel construction and operation.

Specific design parameters such as soil conditions, construction phasing, ventilation, drainage, lighting, pumping, and other mechanical or electrical components should be considered as a part of the design of highway tunnels. For further information on the design of tunnels, refer to **AASHTO's LRFD Road Tunnel Design and Construction Guide Specifications**.

8.2 Rural Design Elements

8.2.1 General Design Considerations

Freeways in rural contexts are comparable to ground-level urban freeways, and they typically utilize higher design speeds and have wider ROW availability. These freeways are generally comprised of four to six through-traffic lanes, the number being dependent on traffic demand and LOS requirements.

The rural town context is generally not applicable to freeways.

Freeways in rural contexts should provide drivers a smooth and generally level ride as these are generally designed for high speed conditions. Rural contexts offer fewer physical constraints, allowing profiles to be generally flat. Gradual adjustments of grade improve sight distance and driver performance, and they also improve the general appearance of the roadway by eliminating sharp elevation changes which may interfere with drivers.

Because freeways in rural areas typically have a wider ROW, wider medians can be utilized to increase driver safety and freeway appearance. Common median width for these freeways range from 50 to 100-ft, with wider medians of 150-ft or more allowing for independent alignments for opposing travel directions. Construction of wide medians also provides space for additional travel lanes that may be needed to handle future traffic volumes.

Rural freeways do not present a large need for frontage roads, as these roads will typically serve only small properties or connection to local roads. Because of this, frontage roads for freeways in rural areas are generally short and sparse.

8.3 Suburban Design Elements

8.3.1 General Design Considerations

Suburban contexts may use a variety of freeway types depending on existing terrain and constraints. These freeways may follow design aspects of both urban and rural freeways.

Suburban contexts may utilize combination type freeways where greater densities exist. Where local streets with higher traffic volumes intersect the freeway, it may be depressed, and the cross street raised to allow both traffic movements proper flow. Longer sections may be designed as ground-level with frontage roads.

Suburban contexts may also contain fewer physical constraints than urban areas, offering wider ROW and less utilities and surrounding buildings. In these areas, crossroads may be more widely spaced. Because of this, it may be common to decrease the cut depth between structures for practical and economic reasons. This simplifies ramp design and can reduce side slopes.

Lane and median width in suburban areas should be designed to meet LOS requirements, see **Section 8.1.5**. Suburban contexts may also make use of special freeway designs such as a reverse-flow freeway or a dual-divided freeway, see **Section 8.4.6**.

8.4 Urban Design Elements

8.4.1 General Design Considerations

Freeways in urban contexts are typically classified as depressed, elevated, ground-level, or a combination thereof. These freeway types are used as appropriate for the surrounding areas or conditions.

Freeways in urban contexts can carry high traffic volumes and may have any number of through-traffic lanes in one direction if the minimum level of service requirements are met. However, it is commonly accepted that freeways become less efficient at widths greater than four lanes in each direction. When more than four lanes in each direction are needed, consideration should be given to implementing a special freeway design. For special freeway designs, see **Section 8.4.6**.

Medians on freeways in urban contexts should be a minimum of 12-ft for a four-lane freeway and 24-ft for a freeway with six or more lanes, and preferably 26-ft with heavy truck traffic to provide a wider shoulder.

Freeways in the urban context typically do not require median crossovers.

8.4.2 Depressed Freeways

Depressed freeways are usually located at a depth of 16-ft or more below the surface of adjacent streets. Major streets pass over this type of freeway, and the depression reduces noise while also allowing surface streets to cross at grade.

Front slopes are not generally used beyond the shoulder of depressed freeways, and in such cases, back slopes should not exceed a slope of 1V:3H. Retaining walls along the freeway should preferably be located a minimum of 2-ft beyond the outer shoulder edge. Minimum median widths should be 10 to 22-ft, and any increase should be in multiples of 12-ft to allow for the addition of more through lanes in the future. Where ROW is extremely limited or costly, walled sections with overhanging frontage roads may be utilized.

8.4.3 Elevated Freeways

Elevated freeways may be used where surface or subsurface ground conditions make a depressed or ground-level freeway infeasible. These can be constructed on either viaducts or embankments depending on current conditions and design requirements.

Elevated freeways allow ground-level traffic to pass underneath, and additional space created beneath the structure can be used for other purposes. These freeways are typically high in cost and are located in dense areas where space is limited. Frontage roads are not typically required due to the elevation difference allowing cross-traffic to operate normally, and an offset of 15 to 20-ft is typically recommended from the structure to surrounding buildings.

8.4.4 Ground Level Freeways

Ground-level freeways are constructed in areas of flat terrain, often adjacent to railroads or bodies of water. These freeways should preferably have one-way frontage roads to provide access to surrounding properties.

Ground-level freeway design should minimize the changes in grade to maintain adequate sight distance. While following the existing profile of natural ground, these freeways can be slightly elevated or depressed to create a smoother drive for motorists.

8.4.5 Combination-Type Freeways

Combination-type freeways use variations of elevated, depressed, and ground-level designs. These freeways may be used where variations in profile and cross sections allow for portions of ground-level freeway combined with sections of either raised or lowered facilities.

8.4.6 Special Freeway Designs

Special freeway designs should be considered when more than four lanes in each direction are needed. Special freeway designs that may be appropriate in some urban area locations:

- Freeways with reverse-flow roadways;
- Dual-divided freeways; and
- Freeways with C-D roadways.

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for more information on special freeway designs.

8.4.6.1 Freeways with Reverse-flow Roadways

A reverse-flow freeway offers two distinct directions of traffic flow as well as a center section which operates in the peak flow direction during peak demand periods. This is usually accomplished by situating a separate reversible roadway within the normal median area. See **Figure 8-12** for an example of a reverse-flow roadway.

8.4.6.2 Dual-divided Freeways

A dual-divided freeway may be designed when more than eight total lanes are needed and the traffic in either direction is distributed equally. This may be a practical solution when widening an existing freeway where existing traffic volumes should not be disrupted.

One benefit is the ability to close one of the directional roadways for maintenance or construction. One disadvantage, however, is the needed width for shoulder area which may impact surrounding property and terrain. **Figure 8-13** provides an example of dual-divided freeway.

8.4.6.3 Freeways with C-D Roads

For Freeways with a C-D road, see **Section 8.1.19**.

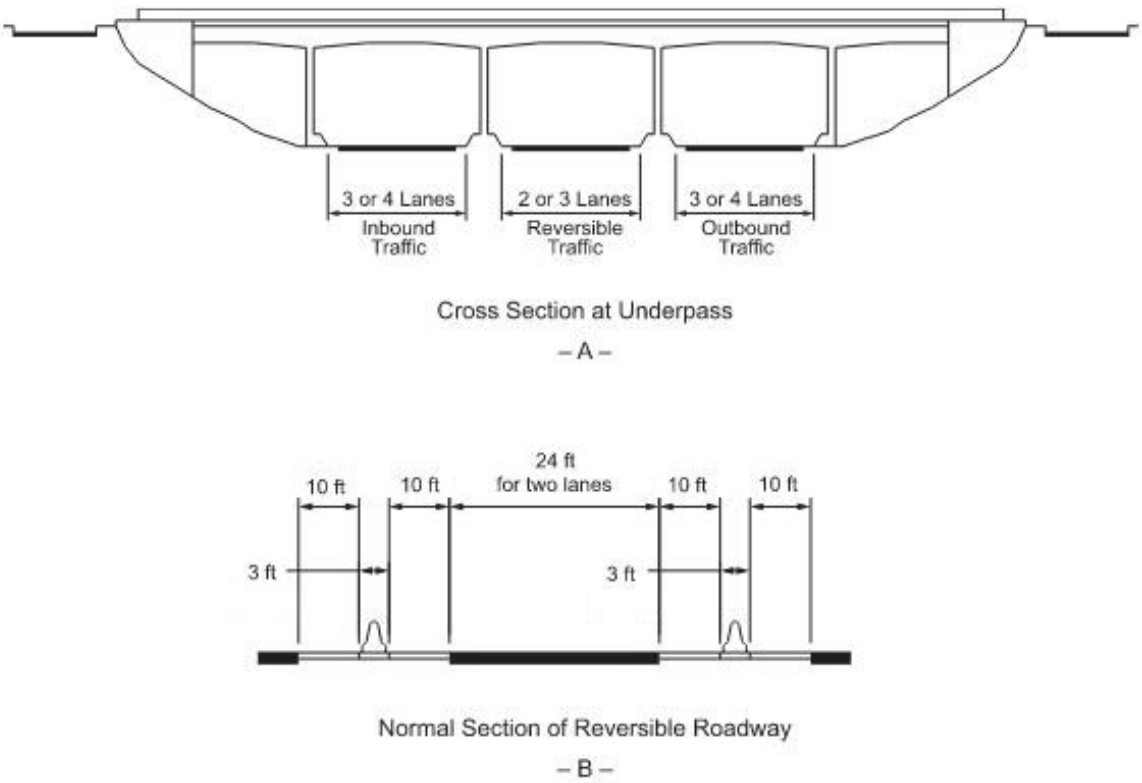


Figure 8-12: Example of Reverse-flow Roadway

Source: AASHTO A Policy on Geometric Design of Highways and Streets



Figure 8-13: Dual-divided Freeway

Source: Virginia DOT

8.5 Urban Core Design Elements

8.5.1 General Design Considerations

Freeways designed in urban core contexts face constraints such as proximity to other structures, high property values, limited ROW, and construction phasing. In these areas, options such as depressed or elevated freeways may serve as the most viable option. During design, the effect on surrounding traffic should be considered. These high-density areas will have increased traffic volumes, and the designer should consider how the construction and resulting freeway might impact these traffic movements.

If significant utility conflicts exist, an elevated freeway could be implemented. This freeway spans above surface streets, allowing local traffic continuity during construction. Space below these freeways can also be used for purposes such as parking where space may otherwise be limited.

Elevated structures may carry one-way or two-way traffic. Where the ROW does not permit a two-way elevated structure, a stacked design may be used. This increases the overall height of the freeway, but the width is minimized.

If conditions permit, a depressed freeway may also serve as a viable option. Lowering the freeway below ground-level can provide freeway traffic to pass below surrounding cross streets while surrounding vehicle and pedestrian traffic flows normally. The lowered freeway also reduces noise which may affect surrounding properties.

For all considerations of special freeway design see **Section 8.4.6** also apply to freeways in the urban core context.

Chapter 9
Mobility Corridor Facilities (5R)

9.1 Design Considerations

9.1.1 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 high-speed, 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.

To support this primary focus of mobility, the design speeds presented in this chapter are between 85-mph to 100-mph. The design criteria may be applied whether planning new facilities or reconstructing existing corridors. While higher operating speeds may not be appropriate in all contexts (such urban and urban core), the 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 (ROW), in terms of maximizing the future use of the corridor.

This does not mean that all projects should 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 (ENV) and the ROW Division (ROW) with questions about environmental and ROW impacts while planning for higher design speeds.

The potential long-term use and growth of the system must be considered and appropriate planning and engineering principles must be applied.

5R projects must be assessed to determine if bicycle accommodations are required per **Chapter 18**; if bicycle facilities are provided, they must meet the additional requirements specified in **Chapter 18**.

The values represented in the following Tables for 5R criteria represent design minimum values and may be increased as project site constraints allow.

This chapter discusses the features and design criteria for mobility corridors and includes the following topics:

9.1 Design Considerations

- 9.1.1 Introduction

9.2 Cross Sectional Elements

- 9.2.1 Overview;
- 9.2.2 Travel Lane Width and Number of Lanes;
- 9.2.3 Shoulders;
- 9.2.4 Pavement and Cross Slope;
- 9.2.5 Vertical Clearance;
- 9.2.6 Stopping Sight Distance;
- 9.2.7 Grades;
- 9.2.8 Horizontal Alignment;
- 9.2.9 Superelevation;
- 9.2.10 Superelevation Transition; and
- 9.2.11 Vertical Curves.

9.3 Roadside Design Criteria

- 9.3.1 Clear Zone;
- 9.3.2 Slopes; and
- 9.3.3 Medians.

9.4 Ramps and Direct Connectors

- 9.4.1 Overview;
- 9.4.2 Design Speed;
- 9.4.3 Lane and Shoulder Widths;
- 9.4.4 Acceleration and Deceleration Lengths;
- 9.4.5 Distance Between Successive Ramps;
- 9.4.6 Grades; and
- 9.4.7 Cross Section and Cross Slope.

9.2 Cross Sectional Elements

9.2.1 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 clearance;
- Stopping sight distance;
- Grades;
- Horizontal alignment;
- Superelevation;
- Superelevation transition; and
- Vertical curves.

Departure from these guidelines is discussed in **Section 1.2**.

9.2.2 Travel Lane Width and Number of Lanes

The lane width of a mobility corridor 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 the **HCM.**

9.2.3 Shoulders

The minimum shoulder width of a mobility corridor is 12-ft. This width applies to both inside and outside shoulders, regardless of the number of mainlanes. Shoulders must be continuously surfaced and be maintained.

9.2.4 Pavement and 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 3 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.

9.2.5 Vertical Clearance

Guidance on Vertical Clearance is presented in **Section 4.8.6**.

9.2.6 Stopping Sight Distance

Stopping sight distance for mobility corridors is calculated using Perception-Reaction Distance and Deceleration Distance. The calculated and design distances are shown in **Table 9-1**. Significant changes from level grade may affect

deceleration distance. Adjustment factors can be found in **Table 9-2**.

Table 9-1: Mobility Corridor Stopping Sight Distance on Level Grade

Design Speed (mph)	Break Reaction Distance[1] (ft)	Braking Distance (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
85	312.4	693.5	1,005.9	1,010
90	330.8	777.5	1,108.3	1,110
95	349.1	866.21	1,215.3	1,220
100	367.5	959.8	1,327.3	1,330
Notes:				
1. Brake reaction distance predicated on a time of 2.5-s; deceleration rate 11.2-ft/s^2.				

Table 9-2: Adjustment to Braking Distance On Grades

Grade	-4%	-3%	-2%	-1%	1%	2%	3%	4%
Adjustment	1.130	1.094	1.061	1.030	0.972	0.946	0.921	0.897

9.2.7 Grades

Undesirable speed differentials between vehicle types suggest that limiting the rate and length of the grades should 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 9-3 summarizes the maximum grade controls in terms of design speed.

Table 9-3: Mobility Corridor Maximum Grades

Type of Terrain	Design Speed (mph)			
	85	90	95	100
Level	3%	3%	3%	3%
Rolling	4%	4%	4%	4%

9.2.8 Horizontal Alignment

Table 9-4 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 only realized at full superelevation and should be avoided unless conditions where limited space places constraints on the horizontal geometry allow no other options. These maximum side forces may be allowed for temporary traffic control during construction or maintenance.

Table 9-4: Mobility Corridor Side Friction Factors and Running Speeds for Horizontal Curves

Design Speed (mph)	Maximum Allowable Friction Factor	Assumed Running Speed (mph)
85	0.07	67
90	0.06	70
95	0.05	75[1]
100	0.04	82[1]
Notes:		
1. Values adjusted up to accommodate application of AASHTO Method 5 calculations.		

9.2.9 Superelevation

Table 9-5 and **Table 9-6** 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 9-5: Minimum Radii and Superelevation Rates[1] for Mobility Corridors, e_{max} = 6%

Superelevation Rate, e (%)	85 mph R (ft)	90 mph R (ft)	95 mph R (ft)	100 mph R (ft)
NC[2,3]	30,104	38,571	50,139	66,667
RC[4,5]	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

Superelevation Rate, e (%)	85 mph R (ft)	90 mph R (ft)	95 mph R (ft)	100 mph R (ft)
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. Refer to 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 minimum radii for normal crown (NC) are computed assuming a cross slope of-2.0% with side friction limited to 0.036, 0.034, 0.032, and 0.030 for 85, 90, 95 and 100 mph respectively. If outer lanes are sloped at -3.0% it is assumed traffic will be at the lower running speeds.
4. 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.
5. For curve radii falling between normal crown and reverse crown, a plane slope across the entire pavement equal to the normal crown should typically be used. A transition from the normal crown to a straight-line cross slope will be needed.

Table 9-6: Minimum Radii and Superelevation Rates[1] for mobility Corridors emax= 8%

Superelevation Rate, e (%)	85 mph R (ft)	90 mph R (ft)	95 mph R (ft)	100 mph R (ft)
NC[2,3]	30,104	38,571	50,139	66,667
RC[4,5]	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

1. Computed using Superelevation Distribution Method 5. Refer to 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 minimum radii for normal crown (NC) are computed assuming a cross slope of-2.0% with side limited to 0.036, 0.034, 0.032, and 0.030 for 85, 90, 95 and 100 mph respectively. If outer lanes are sloped at -3.0% it is assumed traffic will be at the lower running speeds.
4. 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.
5. For curve radii falling between normal crown and reverse crown, a plane slope across the entire pavement equal to the normal crown should typically be used. A transition from the normal crown to a straight-line cross slope will be needed.

9.2.10 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 9-7** 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 9-7: 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

Notes:

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

9.2.11 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 on mobility corridors.**

Minimum K-values are calculated using the same equations as in **Chapter 4, Section 4.8**. Design Ks for both crest and sag vertical curves are shown in **Table 9-8**.

.

Table 9-8: Minimum Design K Values 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.

9.3 Roadside Design Criteria

9.3.1 Clear Zone

The clear zone distances for mobility corridors are shown in **Table 9-9**.

Table 9-9: 5R Clear Zones

Design Speed (mph)	Clear Zone (ft)
85	80
90	80
95	90
100	100

9.3.2 Side Slopes

For safety, it is desirable to design relatively flat areas adjacent to the travel way 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 9-10**.

Particularly difficult terrain may require deviation from these general guide values. Where conditions are favorable, it is preferable to use flatter slopes to enhance roadside safety.

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.

Table 9-10: Earth Fill Slope Rates

Height of Fill	Preferable Slope Rate, Vertical:Horizontal	
	Terrain	
	Level	Rolling[1]
0 – 5-ft	1V:8H	1V:6H
5-ft and over	1V:6H	1V:6H
Notes:		
1. Slope rates should be flatter where practical. Deviation may be permitted for particularly difficult terrain conditions.		

9.3.3 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 the clear zones shown in **Table 9-9**.

9.4 Ramps and Direct Connectors

9.4.1 Overview

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.

9.4.2 Design Speed

Similar to facilities with design speeds of 80-mph or less, ramps on these facilities must 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 upper range ramp/connector design speed.**

Table 9-11 shows the values for ramp/connector design speeds.

Table 9-11: 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

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 for applicable ramp speed and superelevation rate e-max, see Table 4-6, Table 4-7, Table 9-5 and Table 9-6.

9.4.3 Lane and Shoulder Widths

Ramp and direct connector shoulder widths (inside and outside) and lane widths are shown in Table 9-12.

Table 9-12: Ramp and Direct Connector 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)
Notes:			
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).			

9.4.4 Acceleration and Deceleration Lengths

Table 9-13 provides design criteria for exit ramp deceleration lengths. Grade adjustment factors for deceleration lanes are shown in Table 9-15.

Table 9-13: 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	-	-
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
Notes:								
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 9-14 provides design criteria for entrance ramp acceleration. Speed change lane adjustment factors for grade effects are shown in **Table 9-15**. A minimum taper length of 600-ft is suitable for parallel type acceleration lanes for design speeds up to 100-mph. For tapered acceleration lanes, a minimum taper length of design speed (DS) to 1 must be used.

Table 9-14: 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)	Acceleration 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 9-15: Speed Change Lane Adjustment Factors as a Function of a Grade

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 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% Downgrade
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% Downgrade
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

Notes:

1. Ratio in this table multiplied by length of deceleration or acceleration distance in Table 9-13 and Table 9-14.

9.4.5 Distance Between Successive Ramps

The minimum acceptable distance between ramps is dependent upon the merge, diverge, and weaving operations that take place between ramps. The **HCM** 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.

9.4.6 Grades

Grades are associated with the design speed selected for the ramp. Design criteria for design speeds greater than 85-mph can be found in **Table 9-3**. **Section 4.8.1** provides design criteria for speeds less than 85-mph.

9.4.7 Cross Section and Cross Slope

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.

Chapter 10
3R Design Criteria

10.1 Design Considerations

10.1.1 Introduction

Rehabilitation (3R) projects consist of 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 **Section 10.1.5.1** for additional clarification on horizontal and vertical alignment.

Design guidelines for 3R projects have been developed to allow for greater design flexibility.

At the District's option, design values above those presented in this chapter may be used.

3R design decisions should be based on an assessment of the safety and traffic operational performance of the existing road and the cost-effectiveness of potential design improvements. Geometric design improvements should be considered as part of a 3R project in the following situations:

- An analysis of the crash history of the existing road identifies one or more crash patterns that are potentially correctable by a specific design improvement;
- An analysis of the traffic operational level of service (LOS) indicates that the LOS is currently lower than the target LOS for the facility or will become lower than the target LOS within the service life of the planned pavement resurfacing (typically 7 to 12 years); or
- A design improvement would reduce sufficient crashes over its service life to be cost effective (i.e., the anticipated crash reduction benefits over the service life of the project exceed the improvement implementation cost).

With a few exceptions (e.g., see bicycle accommodations below), in the absence of any of the three situations defined above, there is no indication that a design improvement is needed as part of a 3R project, and the existing roadway and roadside geometric features should remain in place. Design or traffic control improvements, in addition to pavement resurfacing, should be considered where anticipated benefits exceed anticipated costs.

For Benefit cost evaluations for 3R projects, spreadsheet tools and methods such as those

identified in the **TSAP**, or Tools 1 and 2 from **NCHRP web-only Document 244** may be used to assess both single design alternatives and comparing several alternatives or combinations of alternatives. These tools generate both a Benefit/Cost (B/C) ratio (including safety benefits) as well as the net present value of benefits. Generally, B/C ratios of 2.0 or higher would represent a significant benefit; B/C ratios of between 2.0 and 1.0 although considered positive; additional sensitivity analyses should be done to determine whether or not the identified upgrades are sufficiently beneficial for the project and TxDOT. B/C values below 1.0 should generally not be considered.

These analyses would serve as the basis for determining whether or not specific upgrades should be made. These analyses would also serve as the basis in the justification for any needed design exceptions or design waivers (if established minimums are not met or not intended to be met) if the Benefit cost analysis indicates that such an improvement to the respective minimum values is not warranted.

These guidelines offer sufficient flexibility to allow 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 18

If bicycle facilities are provided, see **Chapter 18** for requirements.

10.1.2 Freeways and Interstates

In 2022 FHWA issued a final rule, which will allow State DOTs to adopt procedures or design criteria, as approved by FHWA, that enable the State to undertake resurfacing, restoration, and rehabilitation (RRR) projects on freeways, including Interstate highways. Freeway facilities follow the same 3R principles and analyses in this chapter. However, a design exception or design waiver would be needed if the respective 4R freeway minimum criteria (as specified in App. A, or App. B) are not met or not intended to be met. A benefit-cost evaluation would again serve as the basis for any needed design exceptions or design waivers.

10.1.3 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.

10.1.4 Minimum Geometric Design Values

Minimum 3R geometric design guidelines for roadways and frontage roads are provided in Appendix A and Appendix B.

A **design exception** is required for not meeting the minimum values shown in Appendix A tables. A **design waiver** is required for not meeting the minimum values shown in Appendix B tables. See **Section 10.1.5.1** with respect to vertical and horizontal alignment. **Criteria is designated as applicable to high-volume roadways, low-volume roadways, or both. A Benefit cost evaluation should serve as the basis for design exceptions and design waivers** (see **Section 10.1.1**).

10.1.5 Design Values

Where existing highway features comply with the minimum design values given in the respective Appendices, the designer may choose not to modify these features. However, where existing features do not meet these values, the designer may upgrade to the design values shown in this manual following a performance-based approach that uses benefit-cost analysis in design decisions.

These values are intended for use on typical 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.

10.1.5.1 Alignment

Typically, 3R projects will involve minor or no change in vertical or horizontal alignment. However, consider flattening 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. Where the superelevation of an existing curve is less than the design superelevation value of the roadway by more than 1%, the superelevation should be restored. 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.

10.1.5.2 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 roadways that do not meet the minimum 3R design speeds (see **Appendix A** or **Chapter 8** for Freeways), 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 anticipated operating speed where practicable. When the anticipated operating speed cannot be met, additional traffic safety measures should be provided including, but not limited to, enhanced signage, pavement markings and delineation.

10.1.5.3 Side and Backslopes

Roadside flattening should be considered for each 3R project with roadside slopes steeper than 1V:4H where sufficient ROW is available. A benefit cost analysis should be conducted to determine the net benefit and viability of such an upgrade (see **Section 10.1.1**).

10.1.5.4 Lane and Shoulder 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. A benefit cost analysis should be conducted to determine the net benefit and viability of such an upgrade.

10.1.5.5 Bridge Widths

Roadway widths on bridges with curbs are measured to the face of curb. Roadway widths on bridges without curbs are measured to the nominal face of rail.

Refer to TxDOT's **Bridge Railing Manual** and TxDOT **Bridge Railing Standards** for the nominal widths of specific rail types and additional guidance.

10.2 Roadway Tradeoff Guidance and Cross-section Optimization

10.2.1 Overview

Four-lane undivided roadways are predominant in rural areas of Texas and have historically had reduced safety performance compared to other types of rural roadways such as four-lane divided and two-lane roadways. Due to the limited right of way (ROW) usually associated with four-lane undivided roadways, there is not always sufficient room to accommodate a four-lane divided roadway. Additionally, two-lane roadways can significantly improve safety performance by converting these to Super 2's or other two lane derivatives.

Research has been performed to compare the safety and operational benefits of converting four-lane undivided highways and two-lane roadways into other cross section alternatives. The alternative cross sections were analyzed by slightly increasing the overall width depending on available ROW and clear zone needs.

These alternative cross sections include:

- Providing a four-lane undivided with a 4-ft median buffer;
- Providing a four-lane undivided with a two-way left turn lane (TWLTL);
- Converting a two-lane section into a Super 2; or
- Converting a two-lane section into a Super 2 with a two-way left turn lane (TWLTL).

Some of the key factors identified as being influential variables affecting rural highway safety are horizontal curvature, driveway density, shoulder width, and operating speed. These factors should all be considered when determining the appropriate alternative cross section.

Guidance on selecting cross-sections for rural highways is provided in **Table 10-1**; in many cases, use of these proposed cross-sections in a 3R project can minimize the need for roadway widening by reallocating existing pavement width and restriping. The driveway activity index shown in **Table 10-1** is defined as residential driveway equivalents per mile including driveways on both sides of the roadway. The equivalents for industrial and commercial driveways are as follows:

- A single industrial driveway is equivalent to three residential driveways
- A single commercial driveway is equivalent to twelve residential driveways

If it is anticipated that any of the inputs (e.g., AADT) will exceed certain thresholds during the Design Life of the roadway, consideration should be given to utilizing the preferred cross section for the anticipated higher threshold(s). Refer to **TTI Reports 0-7035-R1**; and **0-7136-R1** for complete analysis and research for these types of cross-section optimizations.

Table 10-1: Guidelines for Selecting Rural Cross-Sections

AADT	Driveway Activity Index per Mile [1]	Truck Percentage	Preferred Cross-Section
< 3,000	Any	Any	Two-Lane Undivided/ Two Lanes with TWLTL
3,000 – 15,000	< 30	Any	Super 2
	> 30	< 15%	Super 2 with TWLTL
		15 – 25%	Super 2 with TWLTL
		> 25%	Four Lanes with TWLTL
15,000 – 20,000	< 30	< 15%	Four Lanes with 4-ft Median Buffer[2]
		15 – 25%	Four Lanes with 4-ft Median Buffer[2]
		> 25%	Four Lanes with TWLTL
	> 30	< 15%	Four Lanes with 4—ft Median Buffer[2]
		15 – 25%	Four Lanes with TWLTL
		> 25%	Four Lanes with TWLTL
> 20,000	Any	Any	Four Lanes with TWLTL/ Four-Lane Divided

Notes:

1. Driveway activity index is the number of residential driveways. The index is equal to three times the number of industrial driveways, or 12 times the number of commercial driveways (measured per mile).
2. 6-ft minimum shoulder width. Greater widths are desirable.

10.3 Safety Enhancements

10.3.1 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 does 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.

10.3.2 Safety Design Evaluation

Using a performance-based design approach to inform design decisions, processes and procedures provided in the **HSM**, **TSAP**, and **NCHRP Web-Only "Document 244: Developing Guidelines for Integrating Safety and Cost-Effectiveness into Resurfacing, Restoration, and Rehabilitation (3R) Projects** may be used to estimate the effectiveness of design treatments in reducing crash frequency andseverity, and their cost-effectiveness.

Opportunities to incorporate project specific treatments to potentially reduce crash frequency and severity should be considered early in the 3R design process.

At the beginning of 3R project design, highway designers should assess existing physical and operational conditions related to safety. The following treatments and assessments of these conditions are:

- 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 **CRIS** Comprehensive Safety Dashboard and **CRIS** Crash Trees to identify both targeted treatments and systemic safety countermeasures;
- Conduct a crash analysis. Refer to the **TSAP** and **HSM** for diagnosis procedures to identify causes of collision, safety concerns, and crash patterns. Coordinate with DES-Project Delivery section for further guidance. Additional information is available from the Traffic Safety Division (TRF). 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;
- 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, if this is a natural pathway for wildlife movement;

- 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;
- 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;
- Review the four-year **District Safety Plan** and applicable **Strategic Highway Safety Plan (SHSP)** strategies. This review may identify operational or safety issues to be aware of and corrective measures that may be taken where practicable 3R projects should be developed consistent with the District Safety Plan and SHSP strategies;
- If the analysis of crash history on an existing road identifies one or more crash patterns that are potentially correctable by a specific design improvement or it is anticipated that a safety improvement would reduce sufficient crashes over its service life to be cost effective, designers should consider potential improvements to the site that could be implemented in conjunction with the 3R project and select one or more alternative designs that may reduce the crash frequency and/or severity at the site for further evaluation. Refer to the **HSM** for guidance on selecting countermeasures based on the crash patterns of interest and related crash contributing factors;
- In addition to pavement repairs and geometric improvements, designers should consider incorporating intersection, roadside, and traffic control improvements into 3R projects 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;

- Evaluate guard fence installations at bridge approaches, and existing bridge rails for rehabilitation or replacement;
- Consider approach signing or delineation if appropriate, regardless of whether bridge widening is necessary on a particular project;
- The TxDOT Safer By Design Tool (found on the **Safer by Design webpage** on TxDOT.gov) uses proven safety measures that are based on the predictive methods included in the **HSM**. This tool, however, is unique to Texas roadway facilities and provides quantitative scores that are specific to how a change in a roadway design feature may directly influence the safety performance of a roadway. This tool provides an approach for evaluating potential improvements prior to implementation. A safety score should be determined early in the project development process so that the designer can optimize the proposed safety performance. Refer to TxDOT's **PDP** for additional information on when the TxDOT Safer By Design Tool should be utilized. The tool should also be used anytime future geometric improvements are programmed;
- Before developing construction plans and specifications, designers should document the project evaluation, and the design criteria which will be used. Other evaluation methods not presented here have also been successfully used to identify potential safety problems. These other methods may be used at the designer's option to meet the particular needs of the project;
- The level of safety analysis (i.e., crash analysis and assessment of design alternatives) should be commensurate with the scale of the 3R project. For smaller scale 3R projects, a less detailed safety analysis is sufficient. For larger scale 3R projects, a more detailed safety analysis is recommended;

The TSAP further documents the recommended methodologies and procedures used in Safety and Benefit-Cost evaluations and analyses.

For needed Project Documentation information see **Section 10.7**.

10.3.3 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 **TMUTCD** and the **TxDOT Traffic Standards**;
- Providing a skid-resistant surface; and
- Safety end treating cross drainage pipe culverts 36 inches in diameter or smaller that are inside the respective clear zone.

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. Treatment of trees, utility poles, or other obstacles that are indicated during a crash evaluation should also be considered.

10.3.3.1 Metal Beam Guard Fence

Metal Beam Guard fence must be upgraded to current roadside safety hardware standards. Connections to structures, post spacing, and end treatments must meet current design practices, and standards. See **Chapter 17** for specific guidance on Metal Beam Guard Fence requirements. Metal Beam Guard fence lengths will generally be designed to requirements given in **Section 17.6**.

Remove all nonessential guard fence and guard fence where shielded obstacles may be cost-effectively design treated (removed, made yielding, slope regrading/improvement, etc.).

10.3.3.2 Headwalls

Remove headwalls on small (36 inches or less) cross drainage pipe culverts that are inside the respective clear zone and install sloping (1V:3H or flatter) culvert ends (a.k.a., safety treatments) that blend with existing side slopes. Where located behind guard fence, these culvert ends should be safety end treated and the guard fence removed where there are no other obstructions involved.

10.3.3.3 Cross Drainage Culverts

Cross drainage box and pipe culverts greater than 36-in may remain in place where the clear zones are satisfied. Where the clear zones 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, refer to TxDOT's **HDM** 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 whether they are inside or outside of the clear zone, see **Section 4.12.5** for additional guidance.

10.3.3.4 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.

10.4 Frontage Roads

10.4.1 Overview

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 ROW line to ROW line, even though no additional frontage road lanes are added.

Complete frontage road reconstruction projects, see Chapter 8 and Chapter 15 for the appropriate criteria. Frontage roads are typically classified as a Collector facility.

In some locations, frontage roads are built 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.

Refer to **Appendix A** and **Appendix B** for minimum geometric design criteria of 3R projects on rural and urban freeway frontage roads.

10.5 Bridges & Bridge-Class Culverts (BCCs)

10.5.1 Overview

If a project has a complete bridge replacement or a bridge-class culvert replacement, it is not eligible under 3R permissible scope of work. The 3R criteria will cover certain bridge upgrades, but "no bridge replacement". If a complete bridge replacement is included in a project, then the bridge must follow 4R Design Criteria even though the remainder of the project (such as approaching roadways) falls under 3R criteria. Consult with the Bridge Division as needed for additional information.

Where minimum bridge widths exist, it is generally expected that no additional structural work will be necessary. However, existing conditions such as deficient railing, deteriorated deck, or a structure with an deficient load carrying capability may require additional structural work. In such cases, refer to TxDOT's **Bridge Railing Manual** and consult with BRG 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 TxDOT **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 refer to TxDOT's **Bridge Project Development Manual**.

Note that additional vertical clearance requirements may apply to bridge projects on the Texas Highway Freight Network (THFN), see **Section 4.8.6.**

A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated (An SUP may be used in lieu of the bicycle clear space). 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 **Chapter 12** for specific off-system bridge requirements for current ADT of 400 or less.

10.6 Super 2 Design

10.6.1 Overview

See **Section 10.2.1** on possibility of converting a 2-lane undivided facility to a Super 2 or other configuration.

A Super 2 highway is one in which periodic passing lanes are added to a two-lane rural highway to allow slower vehicles to pass and traffic platoons to disperse. Passing lanes are provided periodically in each direction of travel along the Super 2 corridor, 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 ROW availability and ease of construction. **Figure 10-1** shows nine different configurations of passing lanes. The isolated passing lane shown in **Figure 10-1** a is typically used to reduce delays occurring at a specific isolated bottleneck and is not truly a Super 2 corridor treatment. The other configurations allow some interaction between consecutive passing lanes in opposite directions, and they are used when traffic improvements are needed in both directions of travel over a corridor. The alternating passing lanes shown in **Figure 10-1f** and **Figure 10-1g** can be used when sufficient width is available; **Figure 10-1g** is the typical cross-section for what is commonly described as a 2+1 road. Overlapping passing lanes, shown in **Figure 10-1h** and **Figure 10-1i**, can be used when a passing lane is located on a crest or sag vertical curve, respectively. Side-by-side passing lanes, shown in **Figure 10-1j**, can be used where the location of a passing lane is constrained by nonflexible factors. Those factors include (but are not limited to) obtaining ROW, when heavy traffic is the cause of platooning rather than no-passing zones, and where the need for passing lanes exists in both directions.

Because the Super 2 design allows such a high degree of flexibility, the practitioner has a great deal of latitude in the details of the design and implementation of a Super 2 corridor. This flexibility also means that the practitioner has a responsibility to exercise good engineering judgment in determining which details to include when designing and constructing the passing lanes and any other elements associated with a project that contains a Super 2 component. Many of the design details are consistent with the principles used for any rural highway, but the context of the passing maneuver on an otherwise two-lane highway needs to be considered. **Guidelines for Implementing Super 2 Corridors in Texas** contains information on comparing cross-section alternatives and how to determine which of those alternatives to select for a given

location based on operational, safety, and economic measures.

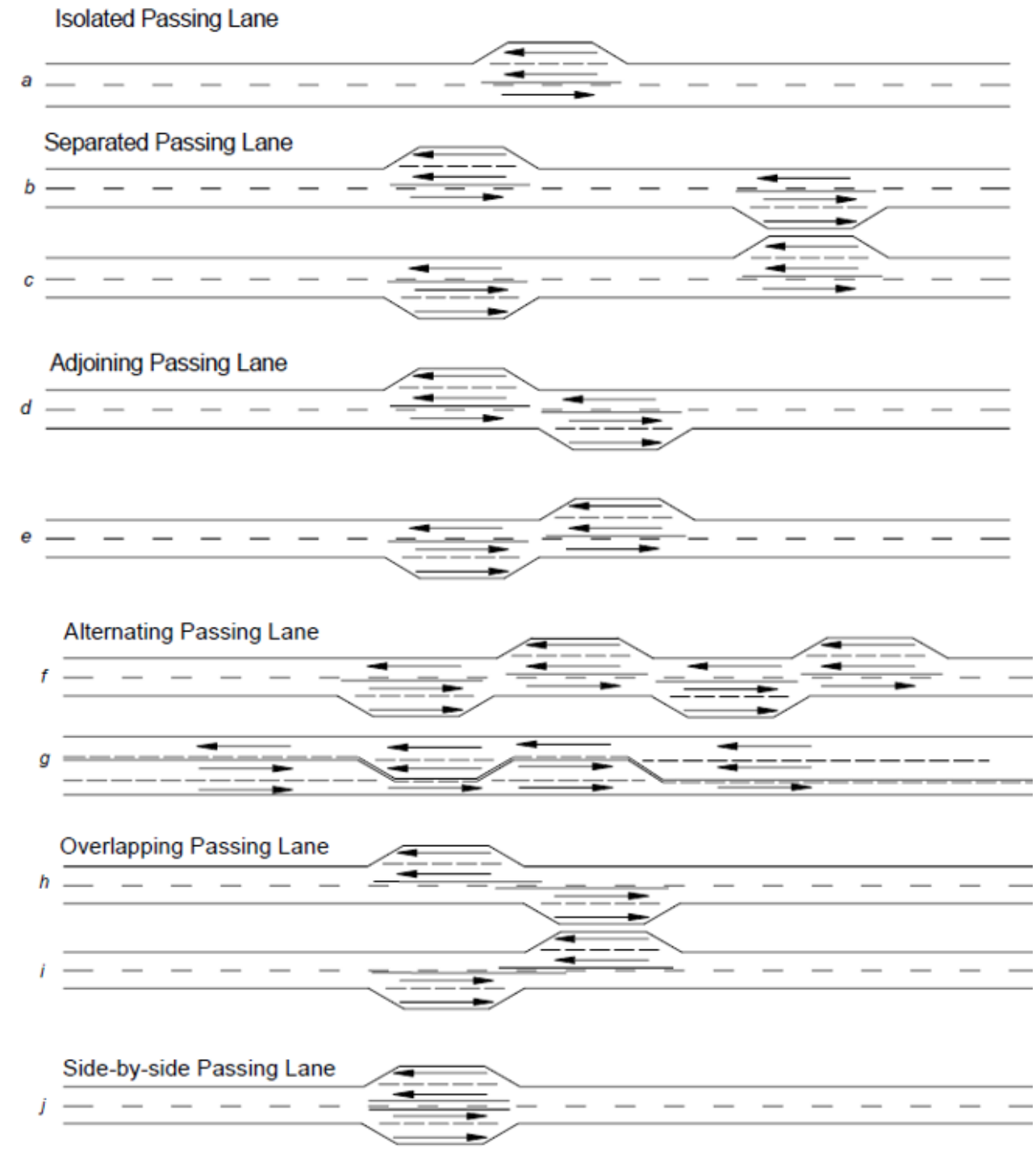


Figure 10-1: Passing Lane Configurations

Source: Mutabazi et al. (1999)

Some issues to consider when designing a Super 2 project:

- Analyze existing ROW 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;
- Provide adequate sight distance (stopping sight distance (SSD) preferable) 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.

Where passing lanes are provided in the vicinity of intersections, appropriate separation between the extents of passing lanes and turning lanes should be provided based on the guidance from **TTI Report 0-7044-R1**. The beginning or end of a passing lane should be at least 1000 ft upstream of an intersection and at least 1500 ft downstream of an intersection to avoid introducing intersection delay associated with the interactions of the minor-road vehicles turning on the major road and those vehicles merging at the end of the passing lane. For guidance from **TTI Report 0-7044-R1** also includes the consideration of appropriate sight distance at the intersection, refer to **AASHTO's A Policy on Geometric Design of Highways and Streets**.

10.6.2 Basic Design Criteria

Recommended design values are shown in **Table 10-2**.

Table 10-2 Design Criteria for Super 2 Highway

	Minimum	Preferable
Design Speed (mph)	See applicable rural two- lane criteria in App. A	See applicable rural multi lane criteria in App. A
Clear Zone (ft)	See applicable rural two- lane criteria in App. B	See applicable rural multi lane criteria in App. B
Lane Width (ft)	11	12
Shoulder Width (ft)	3[1,3]	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 miles is not recommended. Consider switching the direction if more than 4-mi.
3. A 5-ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated (An SUP may be used in lieu of the bicycle clear space). 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 Chapter 12 for specific off-system bridge requirements for current ADT of 400 or less.

The length for opening a passing lane (**Figure 10-2**) should be based on the following:

$$L = \frac{WS}{2}$$

The taper length for closing a passing lane (**Figure 10-2**) should be based on:

$$L = WS$$

Where:

L = Length of taper (ft),

W = Lane width (ft), and

S = Posted speed (mph).

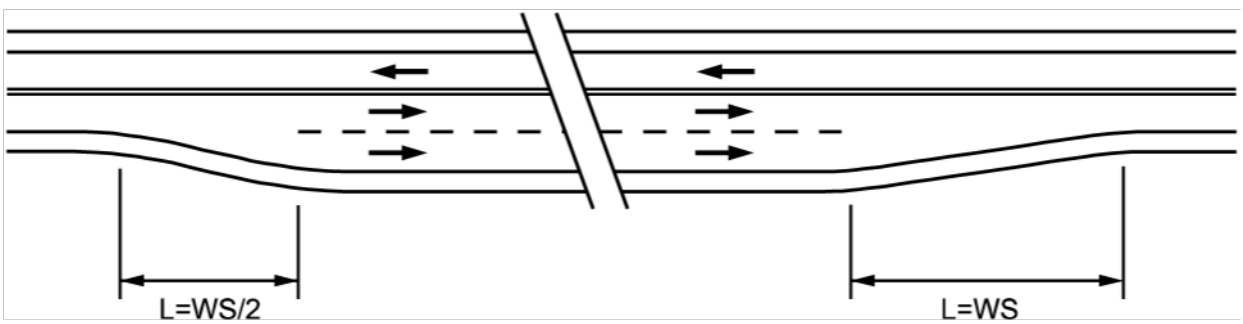


Figure 10-2: Opening and Closing an Outside Passing Lane

When switching the passing lane from one direction to another (closing the inside 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) preferable) between them (**Figure 10-3**).

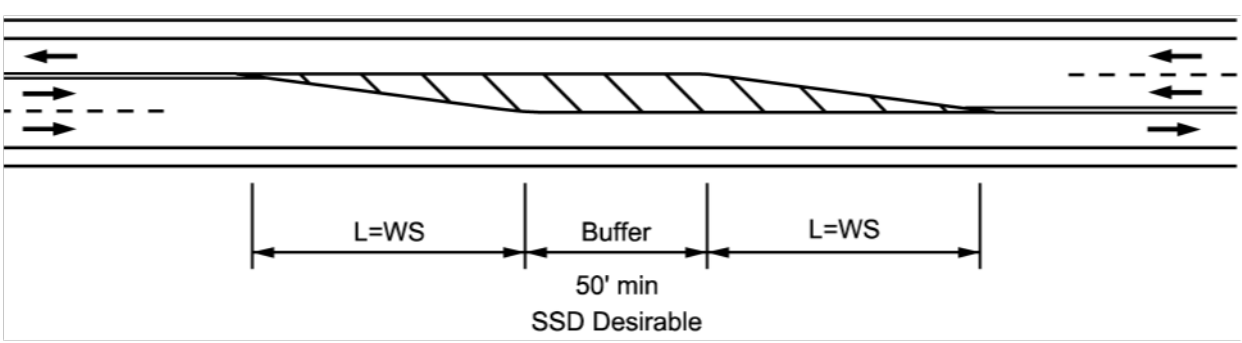


Figure 10-3: Closing an Inside Passing Lane

When opening an inside passing lane in each direction (**Figure 10-4**), provide a taper length based on L= WS/2.

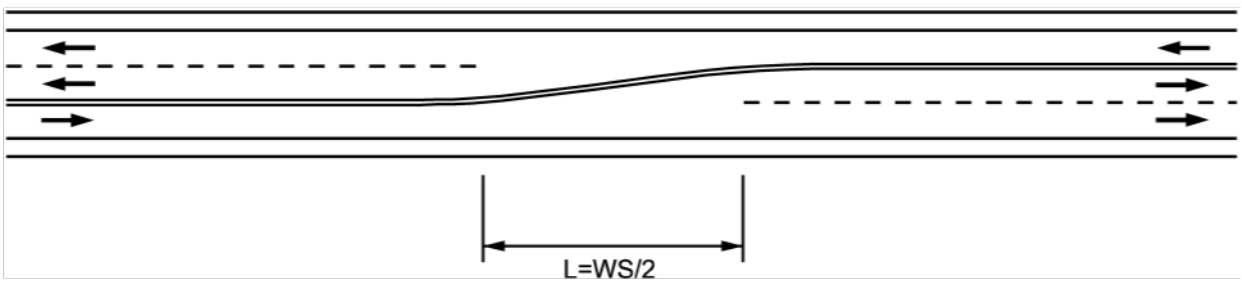


Figure 10-4: Opening an Inside Passing Lane

When widening to the outside of the roadway to provide a passing lane opportunity (**Figure 10-5**), provide an opening taper length based on L = WS/2 and a closing taper length based on L=WS.

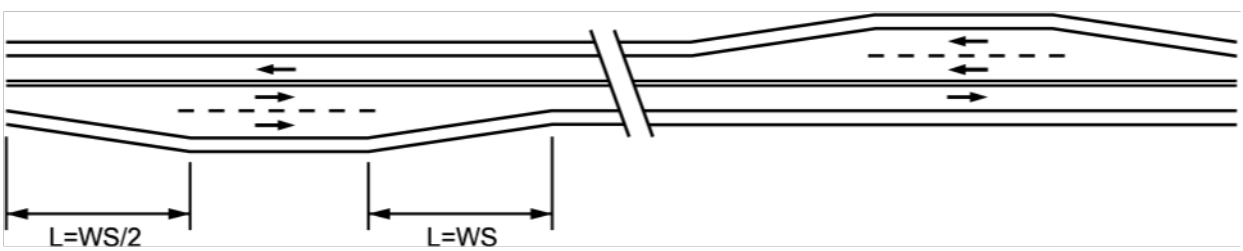


Figure 10-5: Separated Passing Lanes with an Outside Widening

Passing lanes in each direction may overlap if ROW is sufficient (**Figure 10-6**).

Provide an opening taper length based on L = WS/2 and a closing taper length based on L = WS.

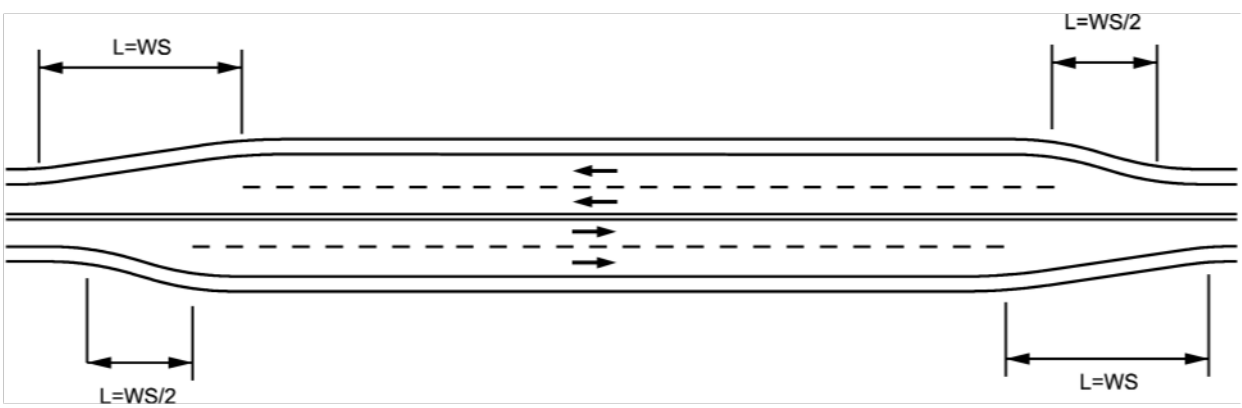


Figure 10-6: Side by Side Passing Lanes

10.7 3R Project Documentation

10.7.1 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 the following:

- 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 countermeasures were considered to address crash patterns of interest and related crash contributing factors?
- What project design alternatives were considered for further evaluation and were **HSM** Part C predictive methods used to estimate the predicted and/or expected average crash frequency for each project design alternative?
- Was a benefit-cost analysis conducted to determine whether one or more of the design alternatives is economically justifiable and determine which design alternative is the most cost-effective investment?
- What other non-monetary considerations factored into the selection of the preferred design alternative?
- 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 the following:

- 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, ROW 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?

10.7.2 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 closely follows the existing grade line (e.g., overlaying, widening, etc), then data from previous plans or a summary sheet (tabular presentation) showing horizontal curve data and vertical curve data 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; and
- 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 11
2R Design Criteria

11.1 Introduction

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 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 BRG.

This chapter discusses the features and design criteria for 2R projects and includes the following sections and subsections:

11.1 Introduction;
- 11.1.1 Definition
- 11.1.2 Upgrading; and

11.2 Crash Analysis

11.1.1 Definition

2R projects are defined as work performed to restore pavement structure, ride 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.

Additional TWLTLs, acceleration/deceleration lanes, and turning lanes should meet the minimum widths of 3R criteria as described in **Chapter 10**. The 2R 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 18**.

11.1.2 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.

11.2 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, the **TSAP** lists several methods that have been used successfully to identify potential crash problems.

Refer to **FHWA's Proven Safety Countermeasures** for recommendations on high value, low-cost actions that are eligible for HSIP funding.

Chapter 12
Special Facilities

12.1 Introduction

This chapter covers considerations for facilities with unique applications and design criteria that may not be covered in other chapters.

This chapter discusses the features and design criteria for special facilities and includes the following sections and subsections:

12.1 Introduction

12.2 Off-System Bridge Replacement and Rehabilitation Projects (Low-Volume)

- 12.2.1 Overview; and
- 12.2.2 Design Values.

12.3 Historically Significant Bridges

- 12.3.1 Reference for Procedures

12.4 Texas Parks and Wildlife Department (Park Road (PR) and Park and Wildlife Road (PW)) Projects

- 12.4.1 Working Agreements

12.2 Off-System Bridge Replacement and Rehabilitation Projects (Low-Volume)

12.2.1 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 and corresponding design criteria 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 4**.

12.2.2 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.

- **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, Nominal Face to Nominal Face of Rail**: A minimum width of 24-ft may be used when ALL following conditions apply:
  - Approach roadway width is 24-ft or less;
  - The county or local entity has no near-term plans to widen the approach roadway; and
  - There is no major near-term development anticipated.

  If any of the conditions above are not met, a minimum width of 28-ft is required. If the existing approach roadway width is greater than 28-ft then

the existing approach roadway width would apply.

- **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 Width**:
  - For minimum length of 50-ft adjacent to the bridge end, the approach roadway width should match the clear roadway width across the structure plus additional width to accommodate approach guard fence.
  - An appropriate transition (minimum length 50-ft) to the existing roadway width should be made in the sections of approach roadway located at the federal project extremities.
  - If roadway surfacing is included, a minimum 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 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 **TMUTCD**, and details should be included in the plans.
  - See **Chapter 23** for more information.

12.3 Historically Significant Bridges

12.3.1 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.

12.4 Texas Parks and Wildlife Department (Park Road (PR) and Park and Wildlife Road (PW)) Projects

12.4.1 Working Agreements

According to **Acts 1995, 74th Leg., Ch. 445, §1**, TxDOT 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 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.

Chapter 13
Intersections

13.1 Overview

For the purposes of this chapter, an intersection is defined as the area where two or more on-system roadways join or cross, including the roadway and roadside facilities for traffic movements within the area. An intersection leg is the segment of roadway connecting to the intersection that forms it. The most common intersection configuration is a four-leg intersection at which two-roadways cross one another. Three-leg intersections are also common. **It is recommended that an intersection have no more than four legs**.

For intersecting legs that join private (residential or commercial) and or public (county road and city street) roads see **Chapter 16**.

The three general types of roadway intersection crossings are:

- At grade intersections;
- Grade separations without ramps; and
- Interchanges (i.e., grade separations with ramps).

This chapter deals primarily with design of intersections at-grade. For the other two intersection types, see **Chapter 15**.

Intersections are often the focus of business and community activity and the place where system users will share the same travel space. Intersections usually have less capacity than other parts of a roadway facility and are where most traffic conflicts occur. Design criteria for an intersection should be selected that:

- Results in a balanced and cost-effective design;
- Offers anticipated efficient operations;
- Results in low crash frequencies;
- Results in low crash severity; and
- Considers the needs of all intersection user groups.

The specific dimensional design criteria presented in this chapter are intended to be used as a guide for the new construction of intersections.

Projects that involve improving existing intersections differ from new construction in that the performance of the existing intersection is known and can guide the design process.

Additional information on the operational and safety analysis of intersections are in the **TSAP**.

13.2 Design Considerations

13.2.1 Intersection Characteristics

An intersection includes the areas needed for all modes of travel that use the intersection. These modes of travel include:

- Bicycle/Micromobility;
- Wheelebase (WB);
- Pedestrian;
- Truck (WB Trucks, Recreational Vehicles, etc.); and
- Transit (School Bus, City Bus, Trains, etc.).

Therefore, the intersection design not only addresses the roadway pavement, but the adjacent sidewalks, pedestrian curb ramps, bicycle facilities, speed-change/auxiliary lanes, medians, and islands. Intersections are a key feature of roadway design in four respects:

- **Focus of Land Use Activity –** The land near intersections often contains a concentration of travel destinations that are accessed by multiple modes of transportation. This is especially true for intersections in the Urban and Urban Core Contexts.
- **Conflict Points –** Pedestrians, bicyclists, and motor vehicles often cross paths at intersections where through and turning movements conflict. These crossings are referred to as "conflict points" and can be further categorized by movement type and severity.
- **Traffic Control –** Movement of users may be assigned through use of traffic control devices such as yield signs, stop signs, and traffic signals.
- **Capacity –** Traffic control at intersections will often limit the number of users that can be accommodated within a given time period on intersecting roadways.

Figure 13-1 shows the number and type of motor-vehicle conflict points at typical four-leg, three-leg, and roundabout intersections. Conflict points should also be considered when locating driveways along a roadway. Providing separation between driveways reduces the potential for collisions by reducing the number of conflict points and increasing the distance between conflict points. See **Chapter 16** and refer to TxDOT's **AMM** for additional information on driveway spacing requirements and driveway design.

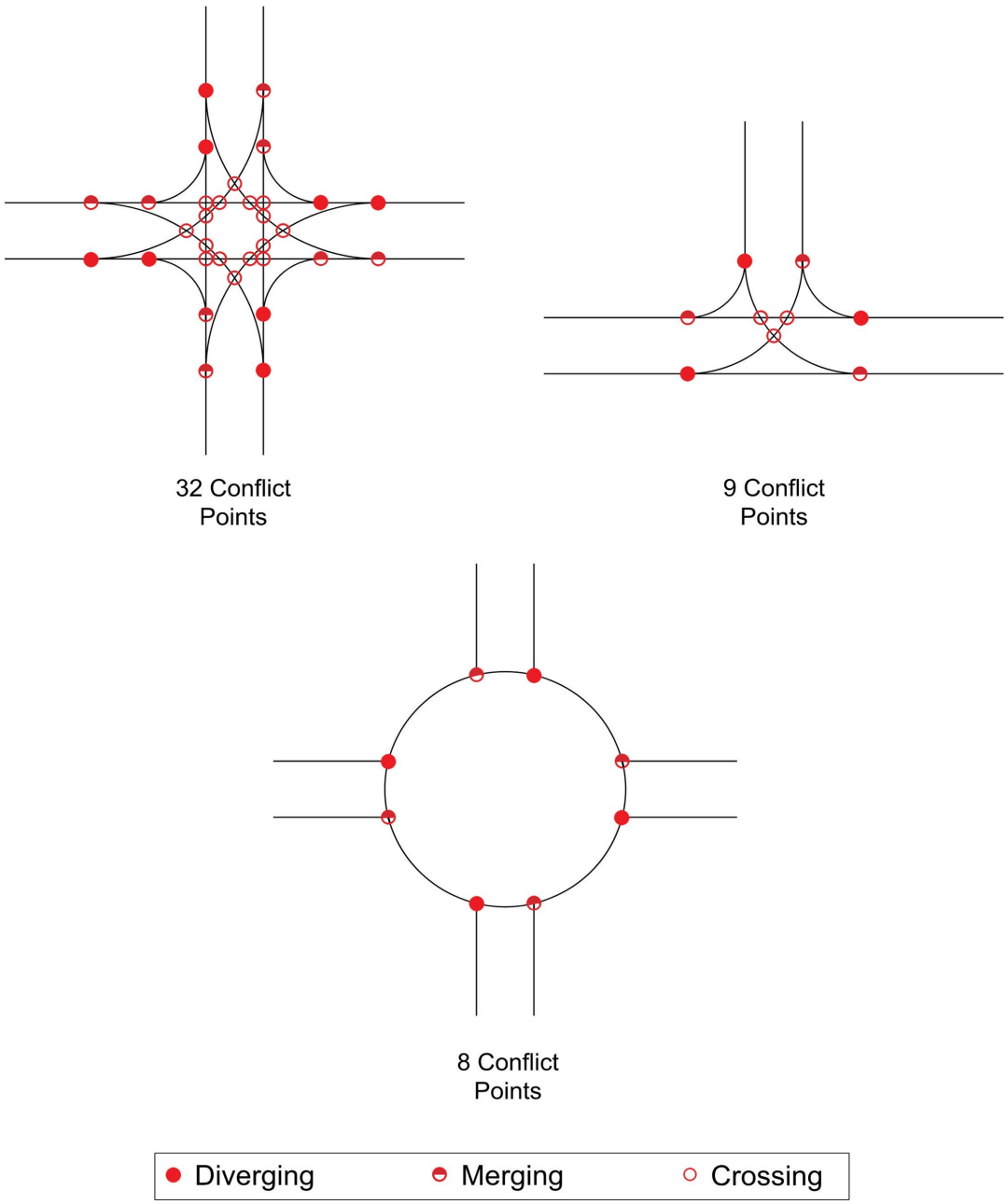


Figure 13-1: Conflict Points at Various Intersection Types

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

13.2.2 Intersection Functional Area

An intersection is defined as both its functional and physical areas, as illustrated in **Figure 13-2**. The functional area extends both upstream and downstream from the physical intersection area and include any auxiliary lanes and their associated channelization. The functional area on the approach to an intersection or driveway is comprised of the following three fundamental components:

- perception-reaction distance,
- deceleration distance, and
- storage length as illustrated in **Figure 13-3**.

The perception reaction distance represents the distance traveled while a driver recognizes the upcoming turn lane and prepares for a maneuver. The distance covered during the perception-reaction period is contingent on factors such as vehicle speed, driver attributes, and driver familiarity with the location. Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional information on perception reaction distance and the time values used for each context classification.

In cases where there are dedicated left or right-turn lanes, the deceleration distance encompasses the space required for both deceleration and lane change. In the absence of turn lanes, it entails coming to a smooth stop. The storage length should be sufficient to accommodate the longest queue expected for the majority of the time. Ideally, **driveways should not be situated within the functional area of an intersection**, as demonstrated in **Figure 13-2**, or within the influence region of an adjacent driveway.

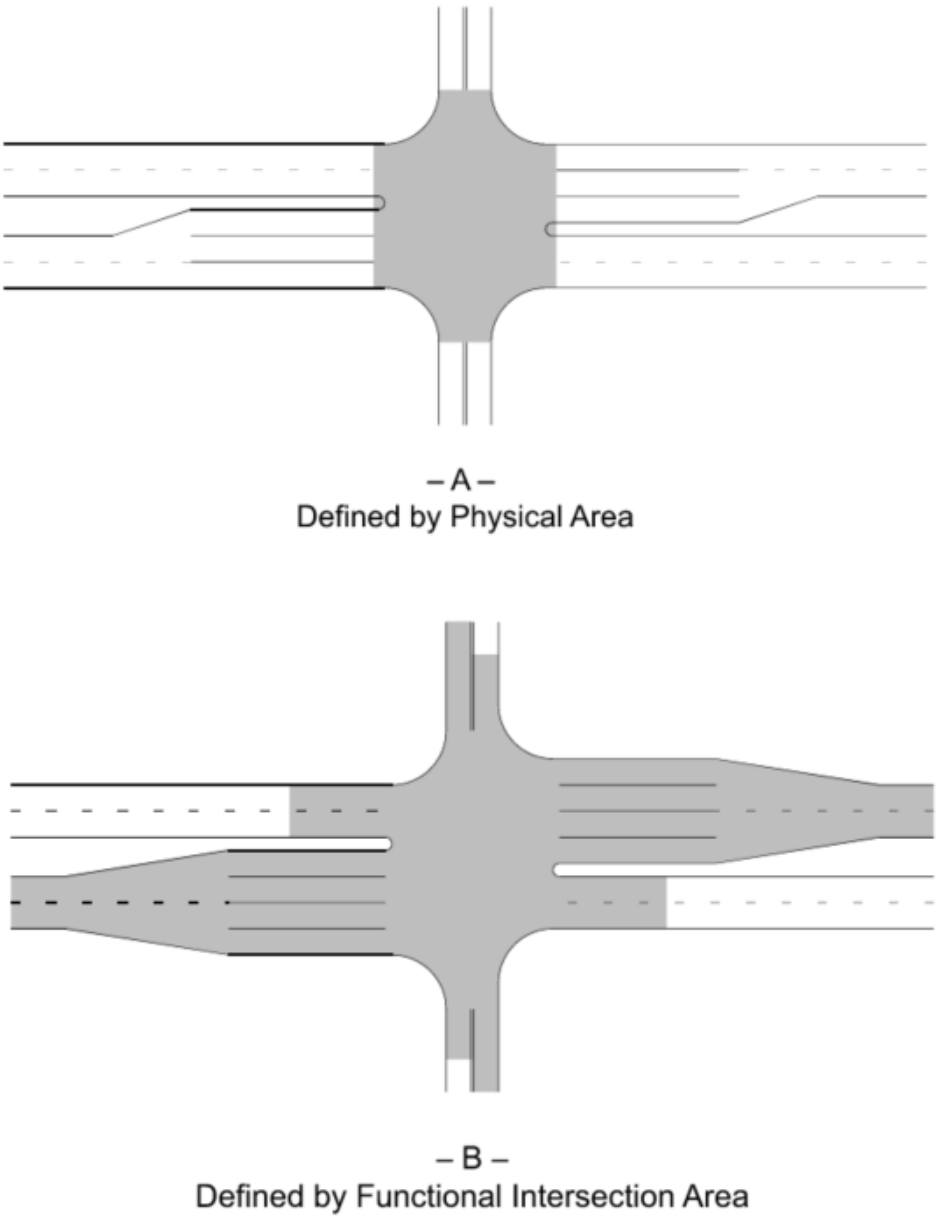


Figure 13-2: Physical and Functional Area of an Intersection

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

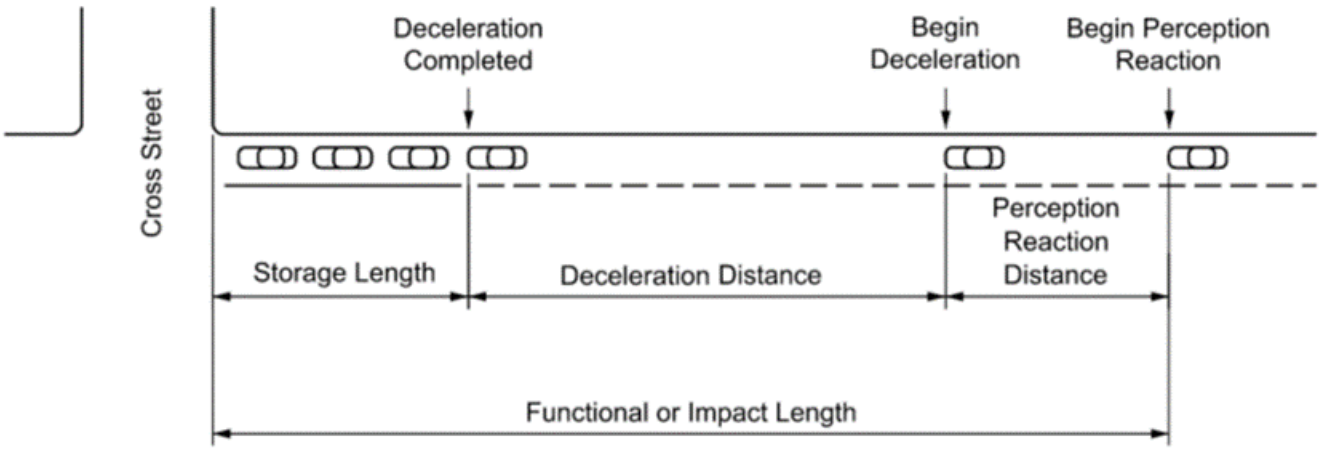


Figure 13-3: Elements of the Functional Area of an Intersection

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

13.2.3 Design Objectives

The goal of any intersection design, regardless of type or location, should be to implement the following principles:

- Reduce vehicle speeds through the intersection as appropriate;
- Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume and lane continuity;
- Provide channelization and appropriate signage that operates smoothly, is intuitive to drivers, and results in vehicles naturally using the intended lanes;
- Provide suitable intersection radii so all turning movements are acceptable for the design vehicle;
- Meet the needs of pedestrians and bicyclists; and
- Provide appropriate sight distance and visibility.

Each element described above influences the operational efficiency and potential for crashes at intersections. When developing an intersection design, the appropriate balance of operational performance for various modes, safety, and cost considerations should be sought throughout the design process. Favoring one component of the design may negatively affect another.

The design of each intersection should achieve an appropriate balance among the competing needs of pedestrians, bicyclists, motor vehicles, and transit with respect to safety, operation efficiency, convenience, ease, and comfort.

Four basic elements should be considered in intersection design:

1. Human Factors

- Driving habits;
- Ability for users to make decisions;
- User expectancy;
- Decision and reaction time; and
- Pedestrian and bicyclist behavior.

2. Traffic Considerations

- Functional Classification of each intersecting roadway;
- Existing and expected future crash frequency and severity;
- Design and actual capacities for all modes of traffic;
- Design-hour turning movements;
- Size and operating characteristics of vehicles and modes;
- Potential conflicts between transportation modes;
- Vehicle Speeds;
- Transit usage and stop locations; and
- Railroad crossing accommodation, if applicable.

3. Physical Elements

- Character and use of abutting property;
- Available right of way (ROW);
- Pedestrian and bicyclist facilities;
- Transit facilities, if applicable;
- Vertical and horizontal alignments at the intersection (see **Section 4.8.5**);
- Sight distance;
- Angle of the intersection;
- Speed change lanes;
- Traffic control devices;
- Lighting;
- Roadside design features;
- Crosswalks;
- Adjacent driveways and driveway spacing;
- Drainage considerations; and
- Provision for utilities.

4. Economic Factors

- Cost of improvements and expected benefits; and
- Cost and effectiveness of controlling access points to abutting residential or commercial properties.

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional design objectives and considerations.

13.2.4 Design Considerations for Intersection User Groups

The design of intersections should use performance measures and engineering judgement to balance the needs of all roadway users and transportation modes. The size and design of physical elements such as roadway

width, lane width, and corner radii are selected according to the volume and priority of each of the intersection user groups.

For an intersection in the urban and urban core contexts, design priority may be given to pedestrians, bicyclists, passenger vehicles, and buses. Basic accommodation may be given to trucks, except for intersections located on truck routes or Freight Network. Intersections in the suburban or rural contexts near industrial and commercial areas may be designed for automobiles and trucks with basic accommodations for pedestrians, cyclists, and transit. Design considerations for users include:

Motor-Vehicles (not including trucks) – Key elements affecting intersection performance for motor vehicles are:

- The type of traffic control;
- Vehicular Capacity of the intersection (i.e., number of lanes);
- Ability and Capacity to make turning movements;
- Visibility of approaching and crossing pedestrians and bicyclists; and
- Visibility of approaching and crossing motor vehicles.

Bicyclists – Key elements affecting intersection performance for bicyclists are:

- The degree to which the roadway pavement is shared or used exclusively by bicyclists;
- The relationship between turning and through movements for motor vehicles and bicyclists;
- Traffic control for bicyclists;
- The differential in speed between motor vehicles and bicycles; and
- Conflicts with pedestrian movements

Pedestrians – Key elements affecting intersection performance for pedestrians are:

- The amount of ROW provided for pedestrians use (e.g., sidewalk width);
- Crossing distance and resulting duration of exposure to motor-vehicle and bicycle traffic;
- The volume of conflicting traffic;
- Speed and visibility of approaching traffic;
- Turning speeds;
- Permissive right-turn on red;
- Permissive left turn movements;
- Accessibility to pedestrians with disabilities; and
- Traffic Control for Pedestrians

Trucks – Trucks share many of the same elements as motor-vehicles described in the preceding sections. In addition:

- Trucks are typically three or four times the length of other motor-vehicles;
- May be much slower starting than most motor-vehicles; and
- May need much larger turning radii than most motor-vehicles.

Therefore, the presence and frequency of trucks affects the capacity of the intersection, the width of the driving surface needed for

turning movements, and the radius of turning movements.

Design of intersection elements for one group of users will often affect other users. For example, an intersection designed to accommodate trucks with no encroachment into adjacent lanes needs large corner radii, wide turning roadways, and results in greater distances for pedestrians to cross.

In addition to the users of the street and intersections, owners and users of adjacent land often have a direct interest in the intersection design. This interest can be particularly sensitive where the intersection is surrounded by retail, commercial, historic, or institutional land uses. The primary concerns include:

- Maintenance of vehicular access to private property;
- Turn restrictions;
- Landscaping;
- Driveway access;
- ROW acquisition; and
- Provision of convenient pedestrian and bicyclist access.

13.2.5 Intersection Capacity

The capacity of a roadway to serve motor vehicles is determined primarily by constraints that are present at intersections. Vehicles turning to and from the primary roadway at unsignalized intersections cause through vehicles to stop or slow, thereby influencing traffic flow and affecting the level of service. The available green time at signalized intersections for any given traffic movement is substantially less than would be available for free-flow operations. Therefore, **it is important to analyze and characterize the combined effects of geometry and traffic control proposed for the design of an intersection**.

Refer to the **TSAP** and the **HCM** for additional information and operational analysis techniques for intersections.

13.3 Types of Intersections

The basic types of intersections are three-leg (T), four-leg, multileg, reduced left-turn conflict, and roundabouts. Further classification of these intersection types includes variations such as unchannelized, flared, and channelized intersections as shown in **Figure 13-4**. **While this figure depicts only vehicle movements, pedestrian and bicycle accommodations should also be included**. See roundabouts in **Section 14.4**.

At each location, the intersection type is determined by:

- Number of intersecting legs;
- Topography;
- ROW constraints;
- The needs of all users;
- Traffic volumes, patterns, and speeds; and
- Desired level-of-service (LOS).

These characteristics are also related to the type of traffic control used. For example:

- Traffic signal;
- Two-way or all-way stop; and
- Yield control on minor approach.

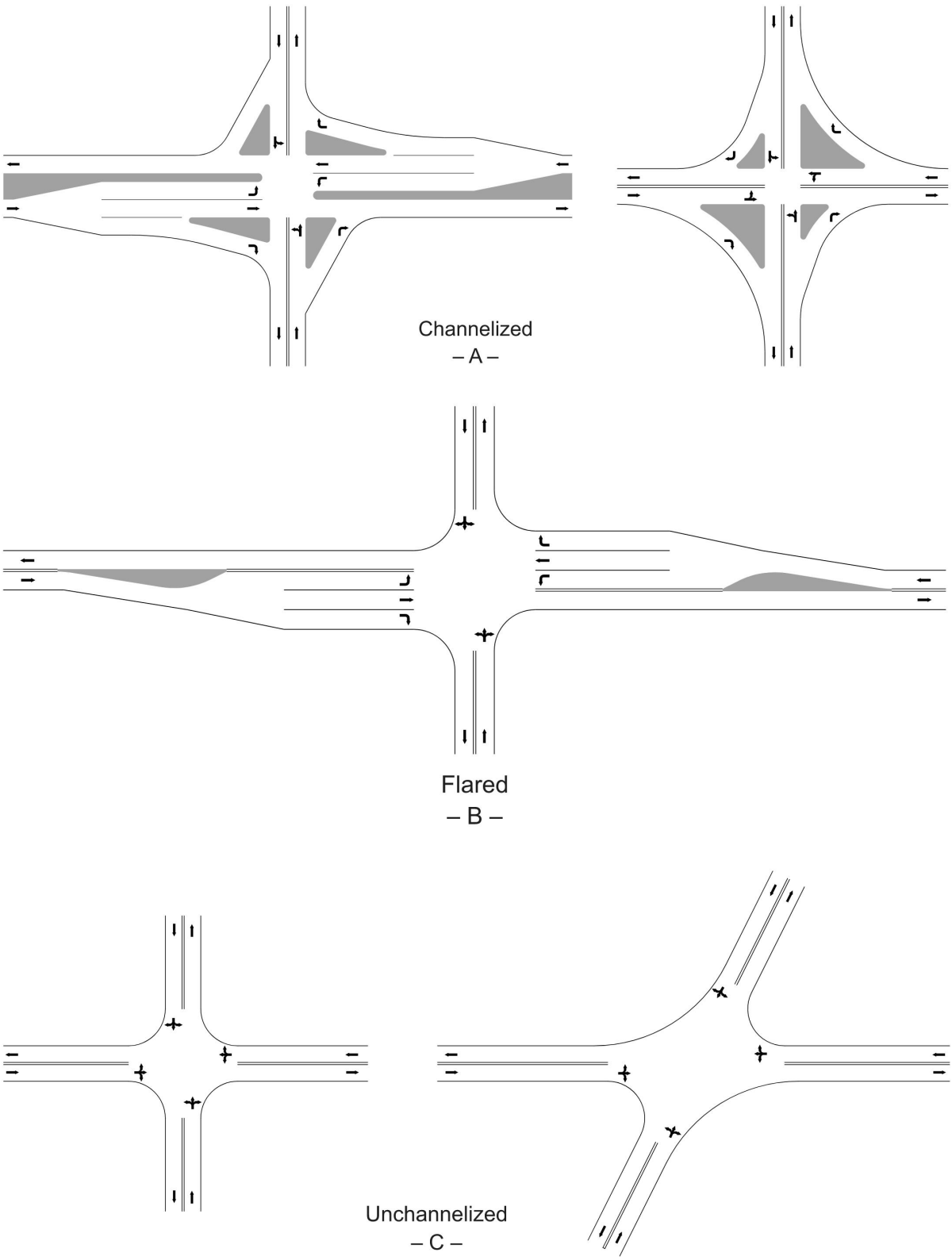


Figure 13-4: General Types of Intersections

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

Any of these intersection types can vary greatly in size, flaring of the pavement for speed-change lanes, and degree of channelization. **Channelization is the separation or regulation of conflicting traffic movements into definite paths of travel**. This is generally accomplished through use of traffic islands or pavement markings to facilitate movements of motor vehicles, bicycles, and pedestrians.

Once the intersection type is established, the design controls and criteria (see **Chapter 4**) should be applied to achieve a proper geometric plan. This section discusses each type of intersection separately with likely variations of each shown. It is not practical to show all possible variations, but those presented in this section are sufficient to illustrate the application of intersection design. For other intersection variations of types and treatment refer to **NCHRP Report 279 Intersection Channelization Design Guide**, which includes detailed examples not included in this section.

While many of the intersection design examples are in urban areas, the principles involved apply equally to design in rural areas. Intersection design needs a balanced approach to accommodate the modes of transportation that are anticipated while also considering the context of the intersection location.

A brief introduction to each intersection type is presented below. Simple intersections are presented first, followed by more complex types. Additionally, the conditions for when each intersection type may be applicable are discussed in the following subsections. See **Section 13.4** for other guidelines to be used in intersection design.

13.3.1 Three-Leg Intersection

13.3.1.1 Basic Types

Basic forms of three-leg intersections (i.e., "T-Intersections") are shown in **Figure 13-5** and **Figure 13-6**. **Figure 13-5(A)** shows the most common type of three-leg intersection. This configuration maintains both roadway widths except for the corner radii where widening is needed to accommodate the turning movements of the selected design vehicle.

This type of unchannelized intersection is usually suitable for junctions of minor or local roads. In rural areas, this type of intersection is commonly used in conjunction with low-volume two-lane highways. In suburban or urban areas, a three-leg intersection may be satisfactory for higher volume two-lane roadways and multilane facilities.

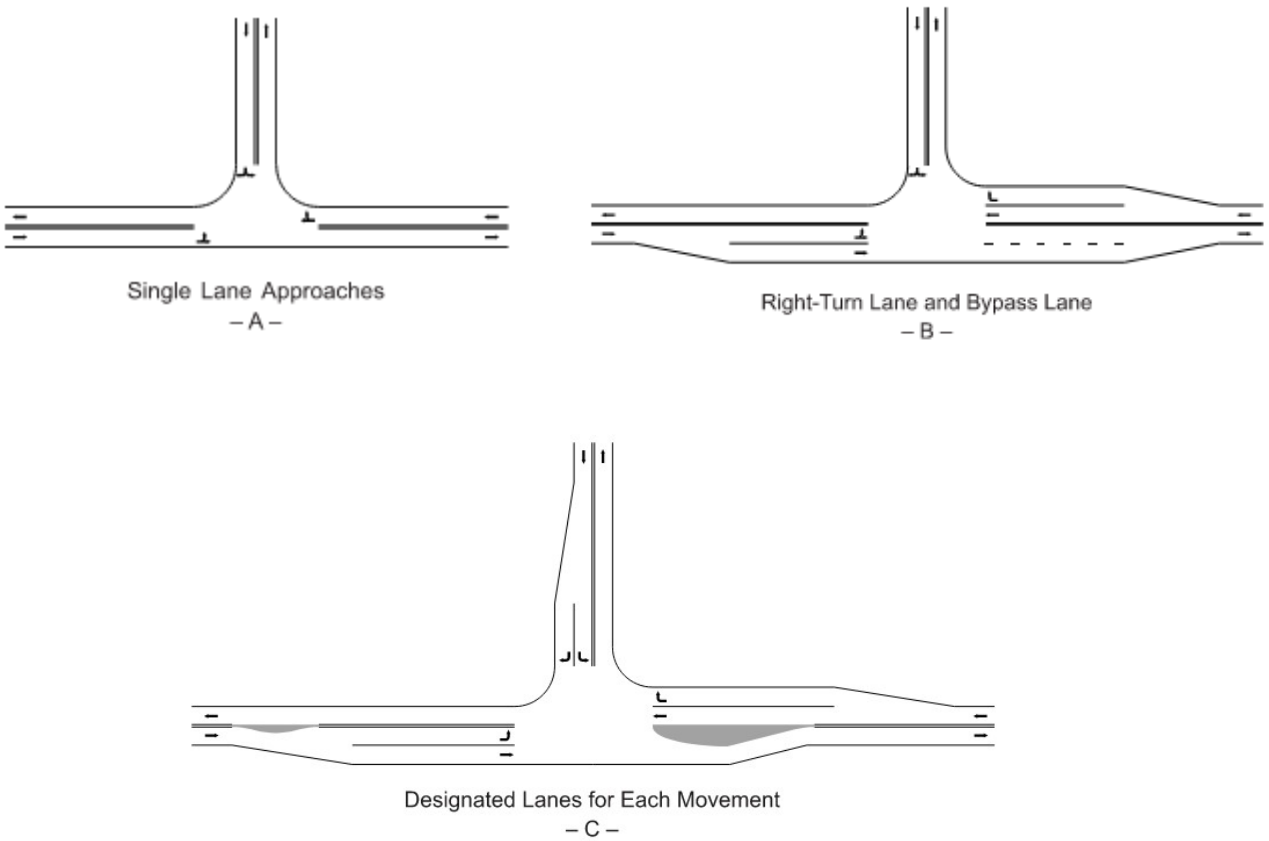


Figure 13-5: Three-leg Intersections

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

Where speeds or turning movements, or both, are high, additional pavement may be provided for speed-change lanes, as shown in **Figure 13-5(B)** and **Figure 13-5(C)**. **However, this type of provision must consider the effects of pedestrian crossing distances**.

The use of speed-change lanes (e.g., left- and right-turn lanes) can:

- Reduce crash frequencies;
- Increase intersection capacity;
- Create better operational conditions for turning vehicles;
- Provide a sheltered storage area for turning vehicles; and
- Reduce speed differentials between through and turning traffic.

13.3.1.2 Channelized Three-Leg Intersections

Where channelization is provided, islands and turning roadways should be designed to accommodate the wheel tracks of the design vehicle movement while providing optimum crossing paths and storage for pedestrians within the proposed intersection.

The simplest form of channelization is accomplished by providing a turning roadway that is separated by an island from the normal traveled ways of the intersecting approaches as shown in **Figure 13-6(A)** and **Figure 13-6(C)**. The approach roadway may include a separated right-turn lane leading up to the turning roadway to accommodate right-turning traffic. Often the provision for a separated left turn lane or through lane to bypass left-turning traffic is appropriate on two-lane roadways where right-turning roadways are also justified. Left-turning traffic can be accommodated by the flaring of the through roadway as shown in **Figure 13-6(B)** and **Figure 13-6(C)**. Right-turning roadways should be designed to discourage wrong-way entry while providing sufficient width for anticipated turning trucks.

Where the traffic demand at an intersection reaches or exceeds the capacity of a two-lane roadway and where signal control may be needed, it may be desirable to convert the two-lane roadway to a divided section through the intersection, as shown in **Figure 13-6(C)**. In addition to adding auxiliary lanes on the through roadway, the intersecting road (i.e., the stem of the three-leg intersection) may also be widened on one or both sides by providing speed change lanes to increase capacity.

Figure 13-5(B) and **Figure 13-6(B)** provide examples of bypass lanes, which are added to the outside edge of the approach, allowing through vehicles to pass left-turning vehicles on the right, while **Figure 13-5(C)** and **Figure 13-6(C)** show traditional left-turn lanes.

Regardless of the treatment, **consideration of traffic demand, delay savings, crash reduction, and construction costs are all key factors in determining whether to install a left-turn lane or a bypass lane**. See **Section 4.10.2** for additional information on bypass lanes.

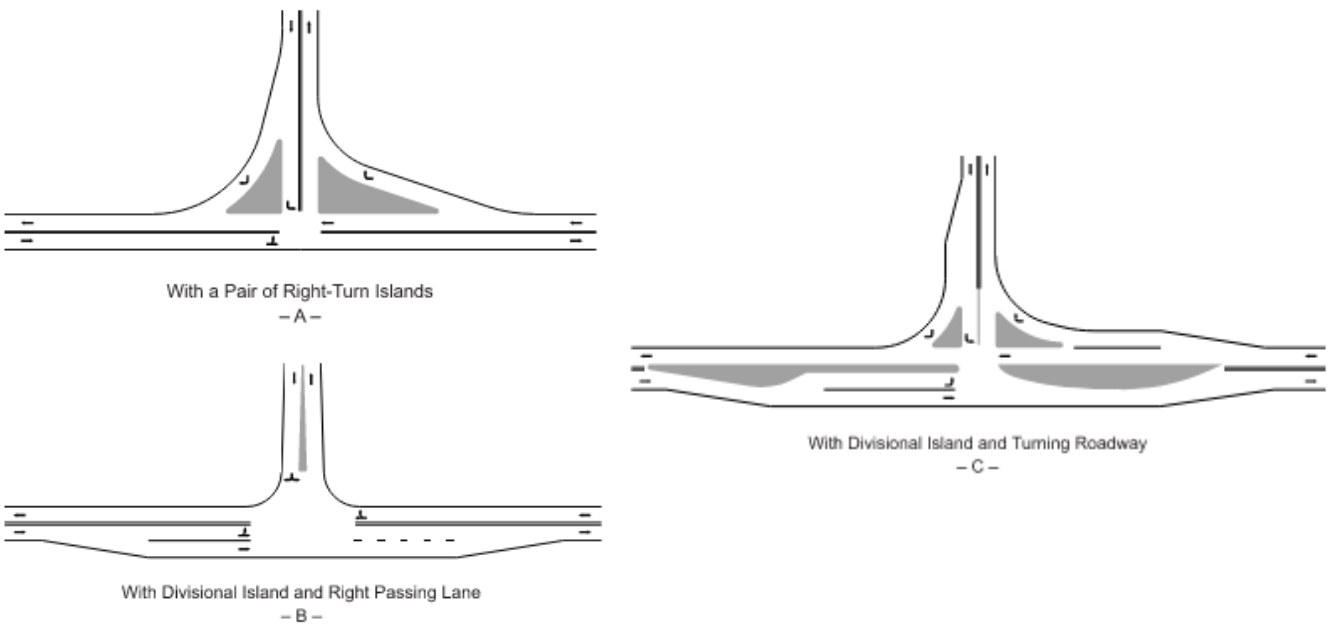


Figure 13-6: Channelized Three-leg Intersections

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

13.3.2 Four-Leg Intersection

13.3.2.1 Basic Types

Basic forms of four-leg intersections are shown in **Figure 13-7** and **Figure 13-8**. Many of the design principles (e.g., island arrangements, use of auxiliary lanes, etc.) of three-leg intersection design apply to four-leg intersection design.

The simplest form of an unchannelized four-leg intersection is shown in **Figure 13-7(A)**. This type of intersection is commonly used for intersections of minor or local roads and often used for intersection of minor roadways with major roadways.

A skewed intersection leg should not be more than 15 degrees from perpendicular.

Flared intersections, shown in **Figure 13-7(B)** and **Figure 13-7(C)**, provide additional capacity for through and turning movements. However, this type of intersection may create concerns for pedestrians due to higher turning speeds and longer crosswalks. Raised medians should be considered for pedestrian refuge.

Parallel auxiliary lanes (i.e., speed change lanes) should be used where traffic volume on the major roadway is near the uninterrupted-flow capacity of the roadway or where through and cross traffic volumes are sufficiently high to warrant signal control. Auxiliary lanes are also desirable for lower volume high-speed conditions.

The length of added pavement should be determined as it is for speed-change lanes (see **Section 4.10.2**).

Figure 13-7(B) shows a flared intersection that provides right-turn lanes on the major highway. This type of configuration is suitable for two-lane roadways where speeds are high, intersections are infrequent, and the left-turning movements from the roadway could create a conflict.

Figure 13-7(C) shows a flared intersection that provides a median lane for left-turn movements. This configuration provides better protection for vehicles turning left from the major highway than the arrangement shown in **Figure 13-7(B)** and is **better suited for intersections with signal control**.

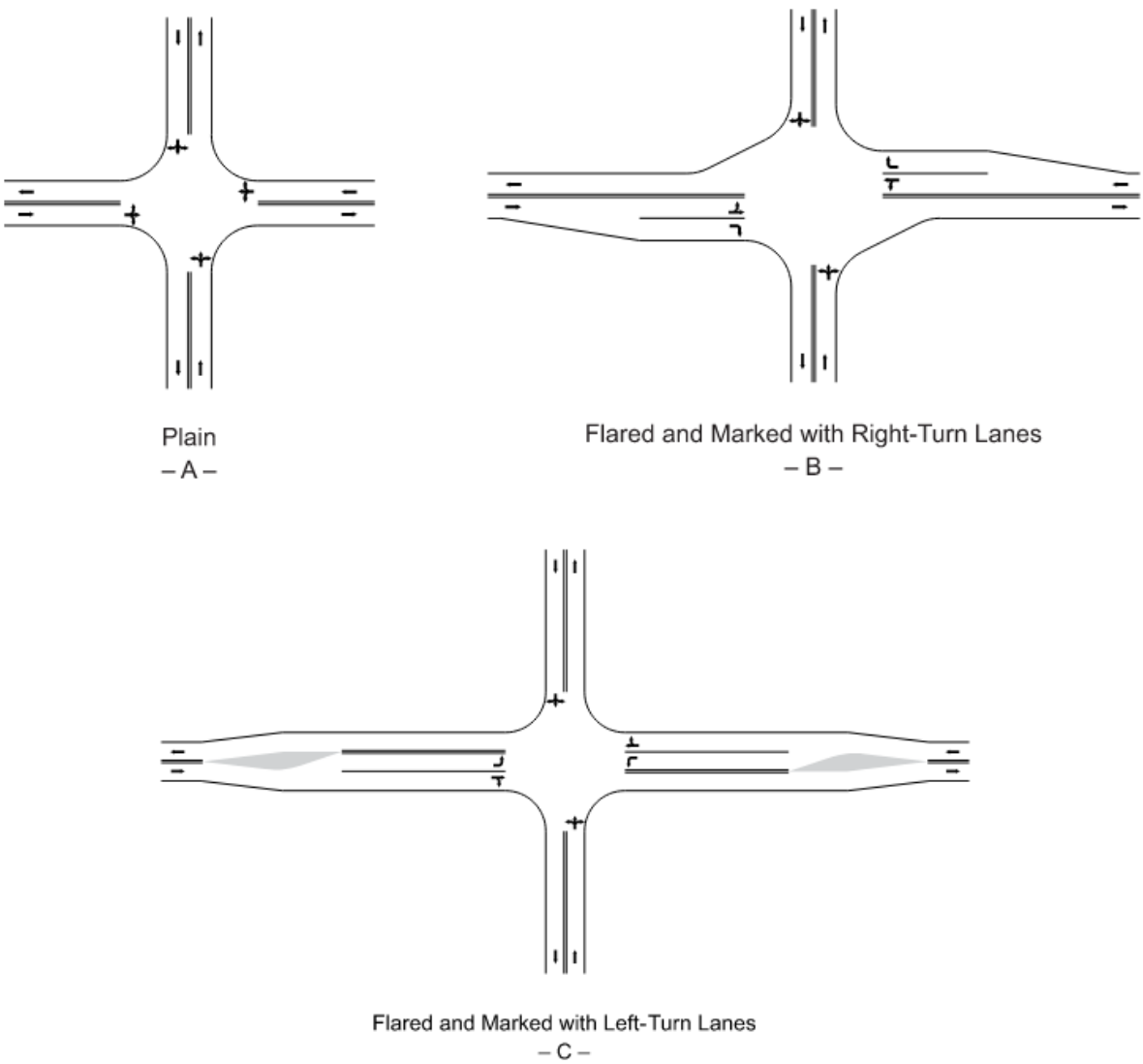


Figure 13-7: Four-leg Intersections

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

13.3.2.2 Channelized Four-Leg Intersections

Typical configurations of four-leg intersections with simple channelization are shown in **Figure 13-8**. Channelized intersections are often provided at major intersection for the more important turning movements, where large vehicles are to be accommodated and at minor intersections in quadrants where the angle of turn is substantially below 90 degrees.

Figure 13-8(A) shows a configuration with right-turn roadways in all four quadrants of the intersection. This configuration is suitable where sufficient space is available and right-turn volumes are high.

Divisional islands on the crossroad intersection are shown in **Figure 13-8(B)** and fits a wide range of volumes. Its capacity is governed by the roadway widths through the intersection.

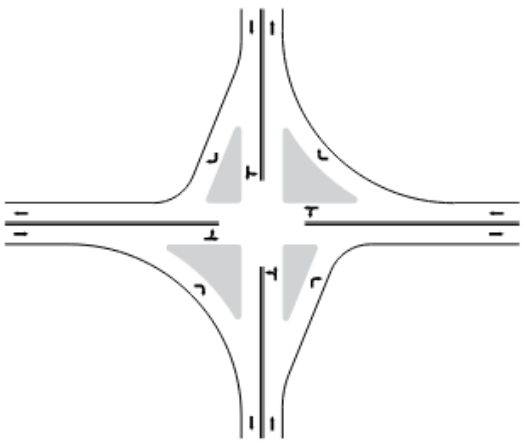
Intersections on two-lane roadways that are operating near capacity or carrying moderate volumes at high speeds should consider a configuration with channelized left-turn lanes as shown in **Figure 13-8(C)**. The auxiliary lanes are used for speed changes, maneuvering, and storage of turning vehicles. The form of channelization on the crossroad should be determined based on the cross and turning volumes and the sizes of vehicles to be accommodated.

Refer to the **August 2023 Federal Register Vol. 88, No. 151 Rules and Regulations**. Crosswalks at multi-lane channelized turn lanes must provide treatments consisting of one or more of the following:

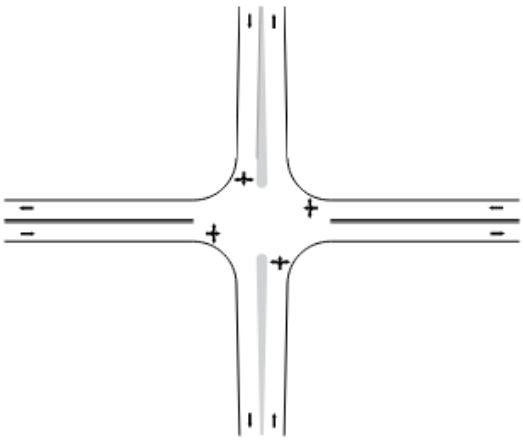
- A traffic control signal with a pedestrian signal head;
- A pedestrian hybrid beacon;
- A pedestrian actuated rectangular rapid flashing beacon; or
- A raised crossing.

Older drivers have difficulty with skewed intersections due to a restricted range of motion and diminished reaction time. Refer to the **FHWA Highway Design Handbook for Older Drivers and Pedestrians** to incorporate the following guidelines:

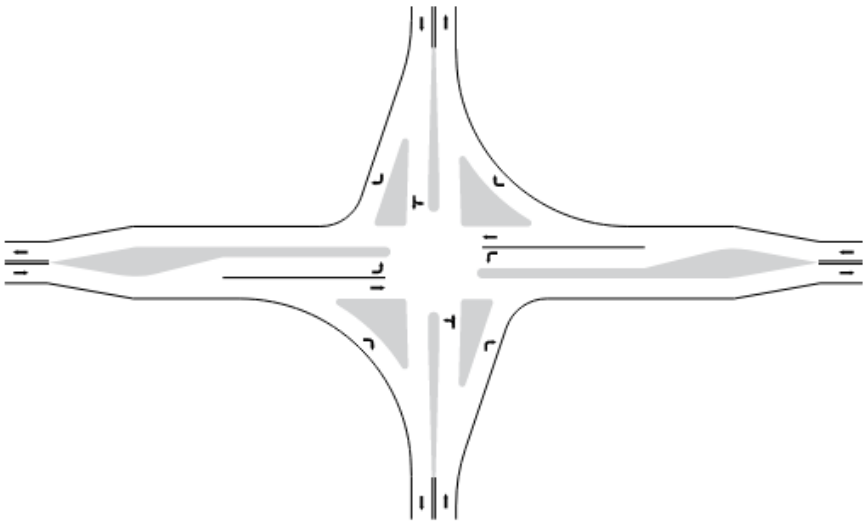
- All intersections should meet at a 90-degree angle (applies to new facilities and redesign of existing facility where ROW is not restricted);
- Intersecting roadway should meet at an angle of no less than 75 degrees in the design of new facilities and the redesign of existing facilities where ROW is restricted; and
- Prohibiting right-turn-on-red is advisable at skewed intersections where the angle between the left approach leg and the driver's approach leg is less than 75 degrees.



With Right-Turn Channelization
– A –



With Divisional Islands
– B –



With Right-Turn Channelization, Divisional Islands, and Left- Turn Lanes
– C –

Figure 13-8: Channelized Four-leg Intersections

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

13.4 Alignment and Profile

The alignment and grade of intersecting roadways should permit all users (i.e., motorists, pedestrians, bicyclists, etc.) to easily recognize the intersection and perform maneuvers needed to pass through the intersection with minimum interference. **The alignment should be as straight and the profile as flat as practical.**

13.4.1 Horizontal Alignment

Intersections should meet at, or nearly at, right angles unless roundabouts are used. When realigning roads at intersections, the greatest benefit is obtained when the curves used to realign the roads allow operating speeds nearly equivalent to the major-roadway approach speeds. **The practice of constructing short-radius horizontal curves on side-road approaches to achieve right-angle intersections should be avoided whenever practical.**

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for more information on realigning roadways at intersections.

13.4.2 Profile

See **Section 4.8.5** Vertical Alignment at Intersections for guidance.

13.5 Intersection Sight Distance

Each intersection has the potential for several different types of vehicular conflicts. The possibility of these conflicts actually occurring can be greatly reduced through the provision of proper sight distances and appropriate traffic controls. The avoidance of conflicts and the efficiency of traffic operations still depend on the judgement, capabilities, and response of each individual driver.

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:

- Minimum intersection sight distance based on the appropriate case should be provided along both highway approaches and across corners (consider the use of landscaping in the sight distance calculations);
- 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.

The provision of adequate sight distance is of utmost importance in the design of 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.

Figure 13-9 shows typical clear sight triangles to the left and to the right for a vehicle approaching an uncontrolled or yield-controlled intersection. **Figure 13-10** shows typical departure sight triangles to the left and to the right of the location of a stopped vehicle on the minor road.

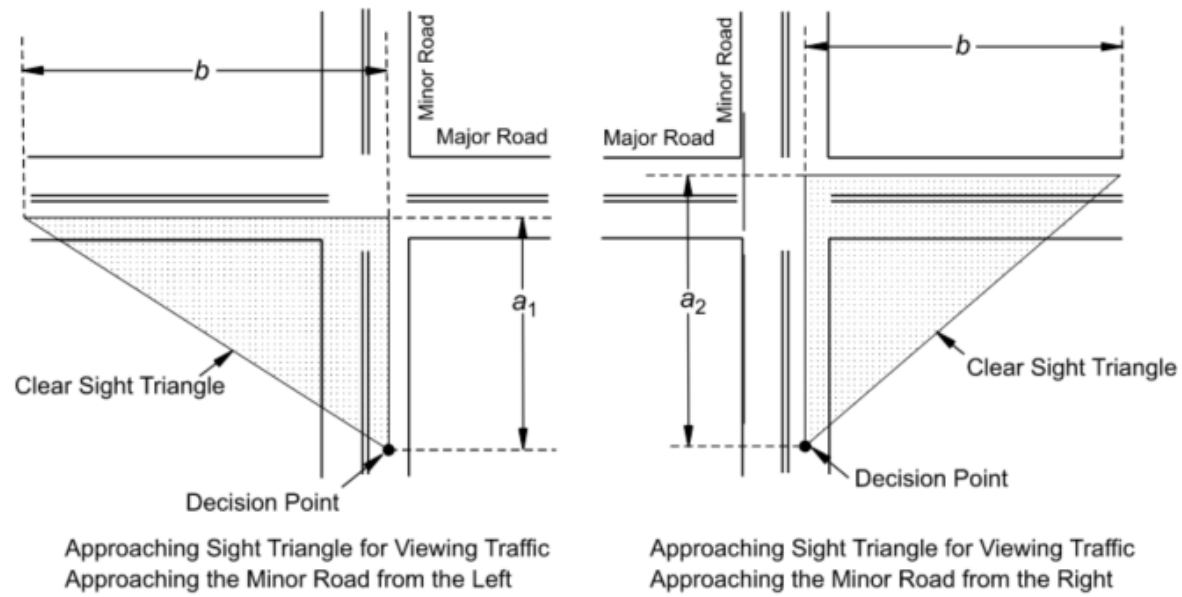


Figure 13-9: Approach Sight Triangles for Intersections

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

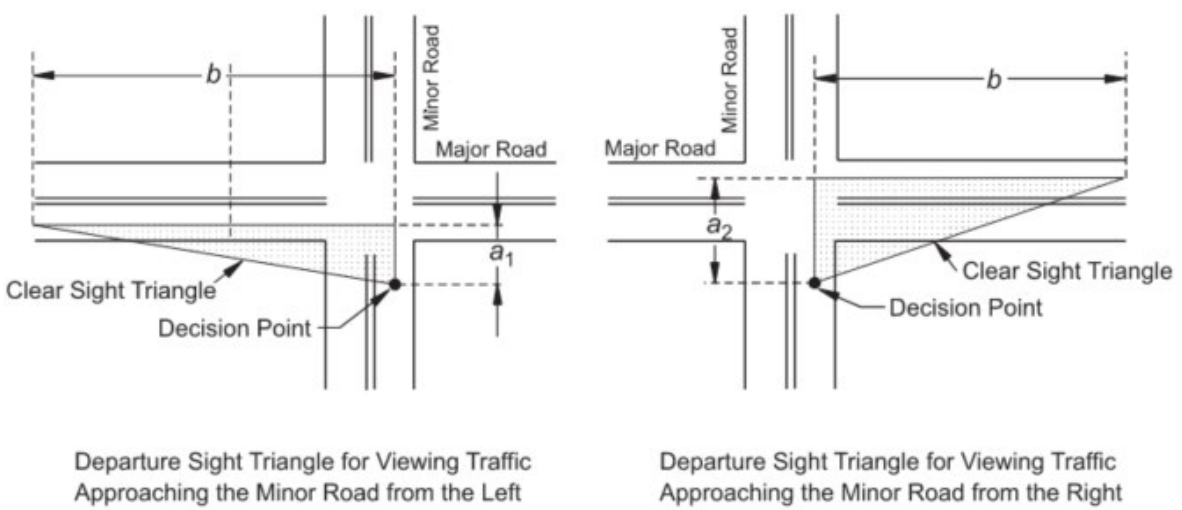


Figure 13-10: Departure Sight Triangles for Intersections

Source: AASHTO's A Policy on Geometric Design of Highways and Streets

Intersection sight distance criteria is provided in the subsequent sections for the following types of intersection controls:

- Case A-Intersections with no control;
- Case B-Intersections with stop control on the minor road;
- Case C-Intersections with yield control on the minor road;
- Case D-Intersections with traffic signal control;
- Case E-Intersections with all-way stop control;
- Case F-Left turns from the major road; and
- Case G-Roundabouts

Intersection sight distance can be determined with the following formula:

$$t_g = t_a + \frac{w + L_a}{0.88V_{minor}}$$

$$b = 1.47V_{major}t_g$$

where:

t_g = travel time to reach and clear the major road (s)

b = length of leg of sight triangle along the major road (ft)

t_a = travel time to reach the major road from the decision point for a vehicle that does not stop (s)

w = width of intersection to be crossed (ft)

L_a = length of design vehicle (ft)

V_{minor} = design speed of minor road (mph)

V_{major} = design speed of major road (mph)

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional information on intersection sight distance.

Table 13-1: Adjustment Factors for Intersection Sight Distance Based on Approach Grade

U.S. Customary														
Approach Grade (%)	Design Speed (mph)													
	15	20	25	30	35	40	45	50	55	60	65	70	75	80
-6	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2
-5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2
-4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1..0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Notes:

1. Based on ratio of stopping sight distance on specified approach grade to stopping sight distance on level terrain.
2. This table is used in determining intersection sight distance criteria for Case C.

13.5.1 Case A-Intersections with no control

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional guidance on Intersections with no control.

13.5.2 Case B-Intersections with stop control on the minor road

Intersections with stop control on minor roads should consider three cases:

- Left turns from minor road;
- Right turns from minor road; and
- Crossing a major road from a minor-road approach.

Intersection sight distance criteria for stop-controlled intersections are longer than stopping sight distance to allow the intersection to operate smoothly. Minor-road vehicle operators can wait until they can proceed safely without forcing a major-road vehicle to stop.

Multiple design vehicles and intersection layout, such as median width, should be considered when determining sight distance. If the design vehicle used for intersection sight distance is larger than a passenger car, then sight distance checks, for both larger vehicle and passenger vehicle, should be calculated.

Left turns from minor roads should use a time gap (t_g)(s) shown in **Table 13-2**. Right turns from a minor road should use a time gap (t_g)(s) shown in **Table 13-3**. Crossing the major road from a minor road approach should use a time gap (t_g)(s) shown in **Table 13-4**.

Table 13-2: Time-Gap for Left Turn from Stop

Design Vehicle	Time Gap (t_g)(s) at Design Speed of Major Road
Passenger car	7.5
Single-unit truck	9.5
Combination truck	11.5
Notes:	
Time gaps are for a stopped vehicle to turn left onto a two-lane highway with no median and with minor road approach grades of 3 percent or less. The time gaps are applicable to determining sight distance to the right in left-turn maneuvers. The table values should be adjusted as follows: *For multilane roadways or medians* – For left turns onto two-way roadways with more than two lanes, including turn lanes, add 0.5 s for passenger cars or 0.7 s for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle. Median widths should be converted to an equivalent number of lanes in applying the 0.5 and 0.7 s criteria presented above. *For minor-road approach grades* – If the approach grade is an upgrade that exceeds 3 percent, add 0.2 s for each percent grade by which the approach grade exceeds zero percent.	

Table 13-3: Time Gap for Right Turn from Stop

Design Vehicle	Time Gap (t_g)(s) at Design Speed of Major Road
Passenger car	6.5
Single-unit truck	8.5
Combination truck	10.5
Notes:	
Time gaps are for a stopped vehicle to turn right onto or to cross a two-lane roadway with no median and with minor-road approach grades of 3 percent or less. The table values should be adjusted as follows: For minor-road approach grades – If the approach grade is an upgrade that exceeds 3 percent, add 0.1 s for each percent grade by which the approach grade exceeds zero percent.	

Table 13-4: Time Gap for Crossing Maneuver from the Minor Road

Design Vehicle	Time Gap (t_g)(s) at Design Speed of Major Road
Passenger car	6.5
Single-unit truck	8.5
Combination truck	10.5
Notes:	
Time gaps are for a stopped vehicle to cross a two-lane highway with no median and with minor-road approach grades of 3 percent or less. The table values should be adjusted as follows: For multilane roadways or medians—For crossing maneuvers that cross roadways with more than two lanes, including turn lanes, add 0.5 s for passenger cars or 0.7 s for trucks for each additional lane, from the left, in excess of two, to be crossed by the turning vehicle. Median widths should be converted to equivalent lanes. For minor-road approach grades—If the approach grade is an upgrade that exceeds 3 percent, add 0.2 s for each percent grade by which the approach grade exceeds zero percent.	

13.5.3 Case C-Intersections with yield control on the minor road

Intersections with yield control on the minor roads should have two separate pairs of approach sight triangles as seen in **Figure 13-9**. A four-leg intersection requires sight triangles for left, right, and crossing maneuvers, while a three-leg intersection only requires left and right sight triangles. Adjustment factors should be applied to intersection sight distance on approach grades based on **Table 13-1**. Where adjustment factors cannot be applied, consideration should be given to installing regulatory speed signing to reduce speed or installing stop signs on one or more approaches.

Intersections with yield control on minor roads should consider two cases:

- Crossing maneuver from the minor road; and
- Left and right-turn maneuvers.

Crossing maneuvers from the minor road should use or exceed a time gap (t_g)(s) shown in **Table 13-3**. Left and right turns should use a time gap (t_g)(s) shown in **Table 13-5**.

Yield-controlled approaches generally need greater sight distance than stop-controlled approaches, especially at four-leg yield-controlled intersections where the sight distance needs of the crossing maneuver should be considered. If sight distance sufficient for yield control is not available, use of a stop sign instead of a yield sign should be considered. In addition, at locations where the recommended sight distance cannot be provided, consideration should be given to installing regulatory speed signing or other traffic control devices at the intersection on the major road to reduce the speeds of approaching vehicles.

Table 13-5: Time Gap for Left or Right Turn at Yield-Controlled Intersections

Design Vehicle	Time Gap (t_g)(s)
Passenger car	8.0
Single-unit truck	10.0
Combination truck	12.0
Notes:	
Time gaps are for a vehicle to turn right or left onto a two-lane roadway with no median. The table values should be adjusted for multilane roadways: For multilane roadways or medians—For left turns onto two-way roadways with more than two lanes, including turn lanes, add 0.5 s for passenger cars or 0.7 s for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle. Median widths should be converted to equivalent lanes. For right turns, no adjustment is needed.	

13.5.4 Case D-Intersections with traffic signal control

Signalized intersections require the first stopped vehicle on one approach to be visible to the first vehicle stopped on all other approaches. Left-turning vehicles should have sufficient sight distance to select gaps in oncoming traffic and complete left turns.

For traffic signal control intersections with a pattern of sight distance-related crashes due to limited visibility, an appropriate crash countermeasure is signalization. However, if the traffic signal is to be placed on two-way flashing operation (i.e., flashing yellow on the major-road approaches and flashing red on the minor-road approaches) under off-peak or nighttime conditions, then Case B sight triangles should be provided for the minor road approaches. In addition, if right turns on a red signal are permitted from any approach, then the appropriate departure sight triangle for Case B should be provided to accommodate right turns from that approach.

13.5.5 Case E-Intersections with all-way stop control

Intersections with all way stop control require the first stopped vehicle on one approach to be visible to the first stopped vehicle on all other approaches. No other sight distance criteria is applicable.

13.5.6 Case F-Left turns from the major road

A left turn from the major road across opposing traffic when permitted, including intersections and driveways, should have sufficient sight distance to accommodate the maneuver. Sight distance design should be based on a left turn by a stopped vehicle since a vehicle that turns left without stopping would need less sight distance.

Left turns from the major road should use a time gap (t_g)(s) shown in **Table 13-6**.

If stopping sight distance has been provided continuously along the major road and if sight distance for Case B (Stop control) or Case C (Yield control) has been provided for each minor-road approach, sight distance will generally be adequate for left turns from the major road. On three-leg intersections or driveways located near a horizontal curve, it is recommended to check the left turn sight distance for possibility of obstructions in the median.

Table 13-6: Time Gap for Left Turns from the Major Road

Design Vehicle	Time Gap (t_g)(s) at Design Speed of Major Road
Passenger car	5.5
Single-unit truck	6.5
Combination truck	7.5
Notes:	
Time gaps are for a stopped vehicle turning left from a two-lane highway with no median For multilane and/or divided roadways—For left turns on two-way roadways across more than one opposing lane, including turn lanes, add 0.5 s for passenger cars or 0.7 s for trucks for each additional lane to be crossed in the left-turn maneuver in excess of one lane. Where the left-turning vehicle must pass through a median, the median width should be converted to an equivalent number of lanes.	

13.5.7 Case G-Roundabouts

See **Section 14.4.5.7**, and refer to **NCHRP Report 1043**, Section 9.5 for intersection sight distance.

13.6 Turning Roadways

Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional guidance on turning roadways and channelization.

13.7 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**. See **Section 4.10.2** for design guidance on speed change lanes.

Where passing lanes are provided in the vicinity of intersections, appropriate separation between the extents of passing lanes and turning lanes should be provided based on the guidance from **TTI Report 0-7044-R1**. The beginning or end of a passing lane should be **at least 1,000 ft upstream** of an intersection and **at least 1,500 ft downstream** of an intersection to avoid introducing intersection delay associated with the interactions of the minor-road vehicles turning on the major road and those vehicles merging at the end of the passing lane. Refer to **TTI Report 0-7044-R1** for guidance that includes the consideration of appropriate sight distance at the intersection, as referred to in **AASHTO's A Policy on Geometric Design of Highways and Streets**.

13.8 Median Openings

See **Section 4.10.15** for guidance on the use of medians at intersections. Also refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional information on design of intersection medians.

13.9 Right Turn Slip Lanes

13.9.1 Urban Design

Right-turn slip lanes at urban intersections must 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. If the slip lane is a multilane crossing for pedestrians, one or more of the following must be provided: a traffic signal, pedestrian hybrid beacon, a rectangular rapid flashing beacon, and/or a raised crosswalk.

The following recommendations address these considerations and are reflected in the configurations shown in **Figure 13-14** (with a deceleration lane) and **Figure 13-15** (without a deceleration lane).

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 it accordingly. **The minimum recommended angle is 55 degrees**.

Curb Radii and Curb-to-Curb (throat) Width - Most 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 **Section 4.3.2**. See **Signage and Pavement Markings** below for lane markings.

Channelizing Island - Islands are recommended to have a minimum side length of 15-ft, excluding the corner radii, refer to **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 cannot be met due to highly constrained conditions. See **Figure 13-11** 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, see **Section 4.10.12** and **4.13.10.1**.

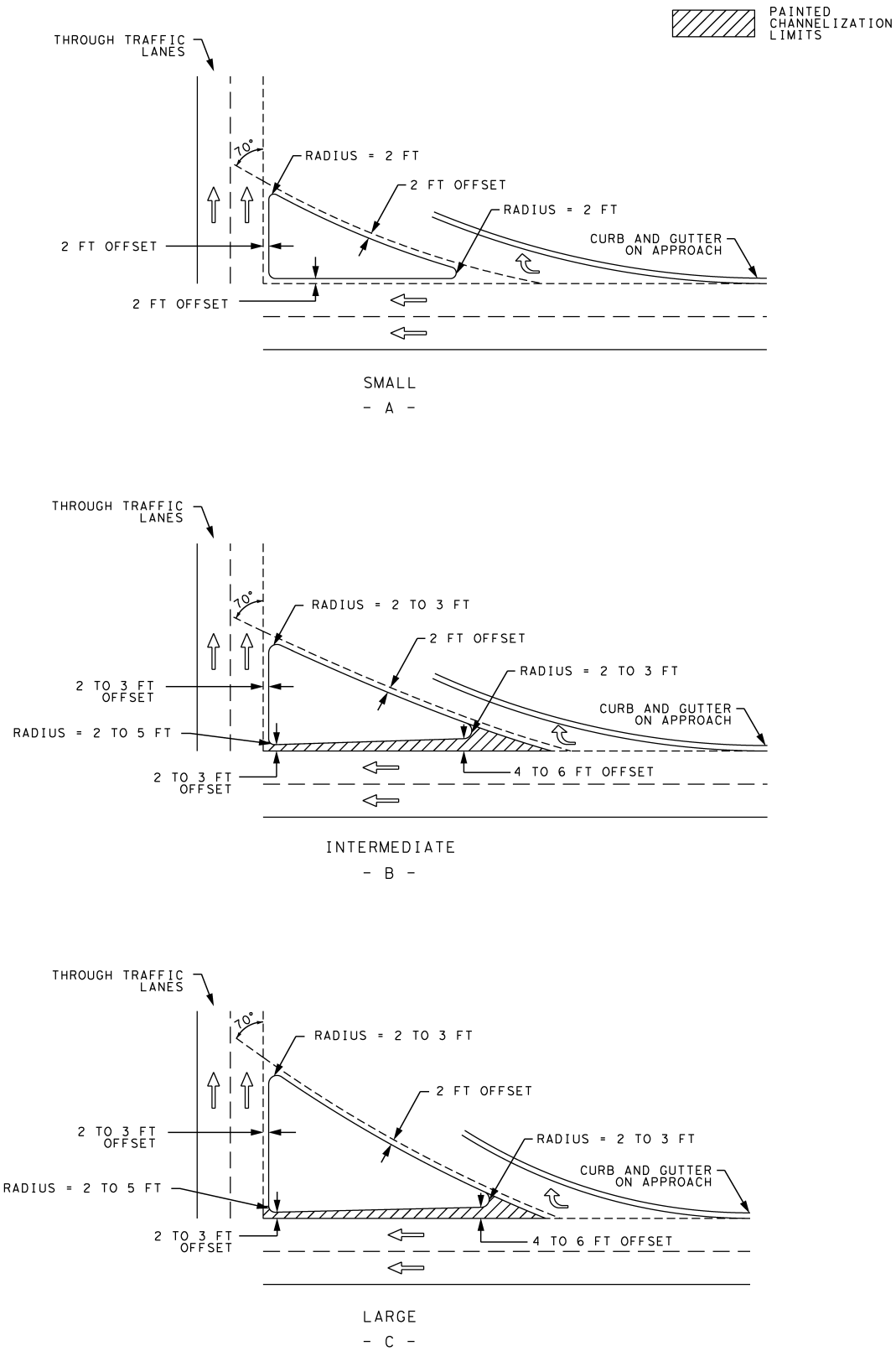


Figure 13-11: Details for Channelizing Island Design for Right-Turn Slip Lane

Pedestrian accommodations are a central component of the design. The pedestrian access routes across channelizing islands are sometimes set at the top of curb elevation using a series of curb ramps and landings (**Figure 13-12(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 13-12(B)**). **This option is preferred** since it is easier for some pedestrians to navigate, especially those with vision disabilities and does provide some wayfinding benefits. However, 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 the roadway gutter grade (**Figure 13-12(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 **Figure 13-12(A)** and **Figure 13-12(C)**.

Provide flared sides where the pedestrian circulation path crosses the curb ramp as shown in **Figure 13-12(A) and Figure 13-12(C).** 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. 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.

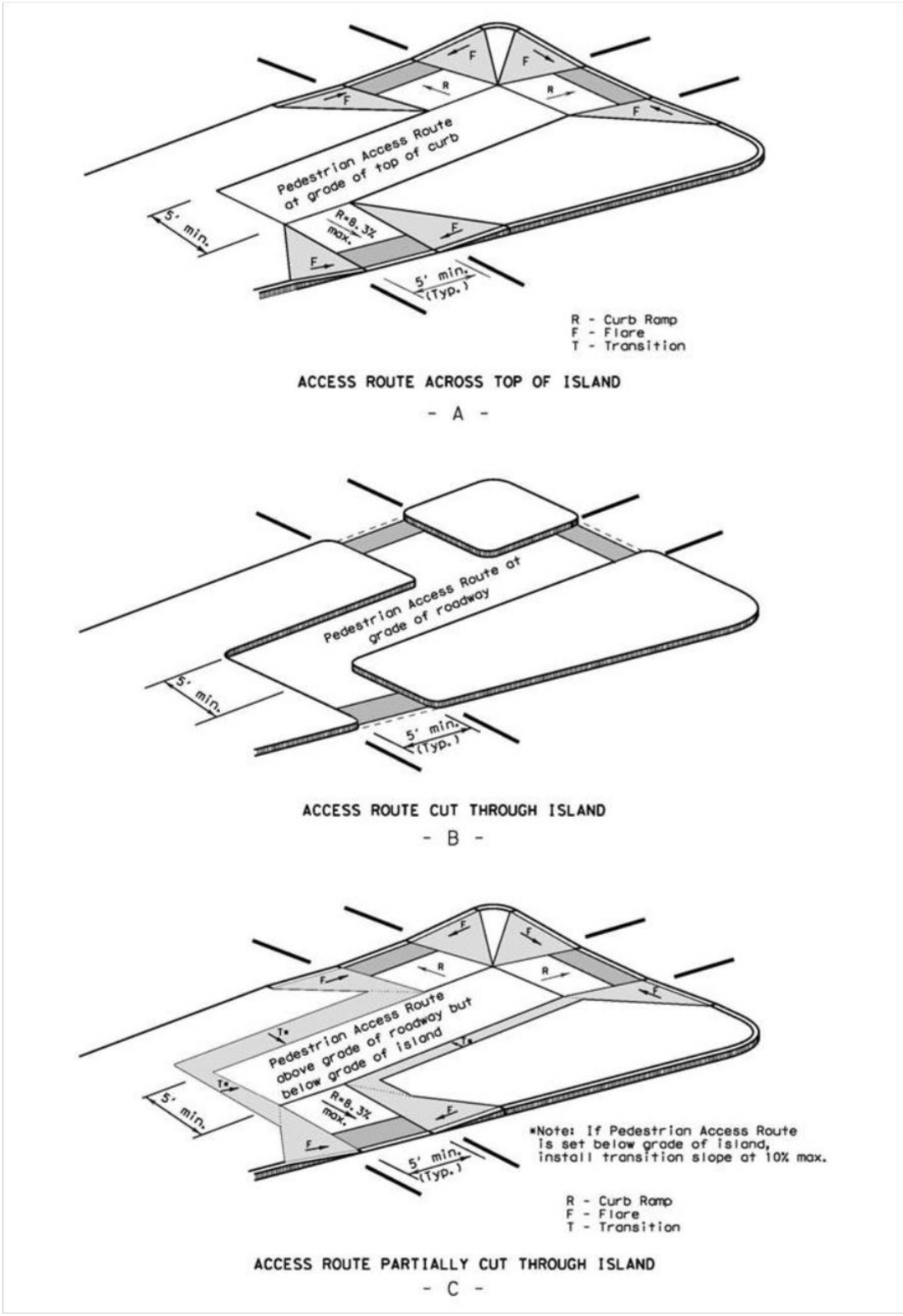


Figure 13-12: Combination Island Ramps (Per Pedestrian Facilities Curb Ramp Standards)

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. Refer to Number, Location, and Spacing of Access Connections in Chapter 2 of the **AMM** for volume thresholds for installing deceleration lanes. See **Section 4.10.2** for design recommendations for deceleration lanes.

If conditions do not necessitate a deceleration lane or ROW is restricted, consideration should be given to using a taper, see **Section 4.10.5**. See **Figure 13-14** and **Figure 13-15** for sample right-turn slip lane designs with and without a deceleration lane.

Acceleration Lane - Acceleration lanes typically are not used on urban streets since:

- They make it more difficult for pedestrians, especially the visually impaired, to cross the turning roadway;
- Cross-street drivers do not expect merging traffic; and
- 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.

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 to minimize paths for water flow. Avoid placing drainage low points at or near the ADA curb ramps.

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. Refer to Chapter 6, of TxDOT's **Highway Illumination Manual** for pole placement guidelines.

Apparatus and Pole Placement - ITS equipment, signal, utility poles, and apparatuses should be outside of paved pedestrian walkways and landing areas. Refer to the **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.

Crosswalk Location - Crosswalks should be placed toward the middle of the channelized island with a **minimum of 20-ft between the crosswalk and yield line** for the intersecting street as shown in **Figure 13-14** and **Figure 13-15**. 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.

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.

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.

Signage and Pavement Markings - Yield signs are typically the appropriate traffic 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 Traffic Standards** 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 **Figure 13-13**).



Figure 13-13: Supplemental Crosswalk Signs

Source: W11-2 and W16-7P 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 **Figure 13-14** and **Figure 13-15**. A raised truck apron may also be considered in the void area to further enhance the channelization of passenger vehicles.

Bicycle Lane - When a bicycle lane is used, it should be striped appropriately to define ROW and shared spaces. Refer to Chapter 18, the **AASHTO Guide for the Development of Bicycle Facilities (AASHTO Bike Guide)**, as shown in the **TMUTCD,** and as detailed on **TxDOT Traffic Standards** for bicycle lanes. Bicyclists intending to make a right turn can use the right-turn lane/turning roadway and operate like a motorized vehicle.

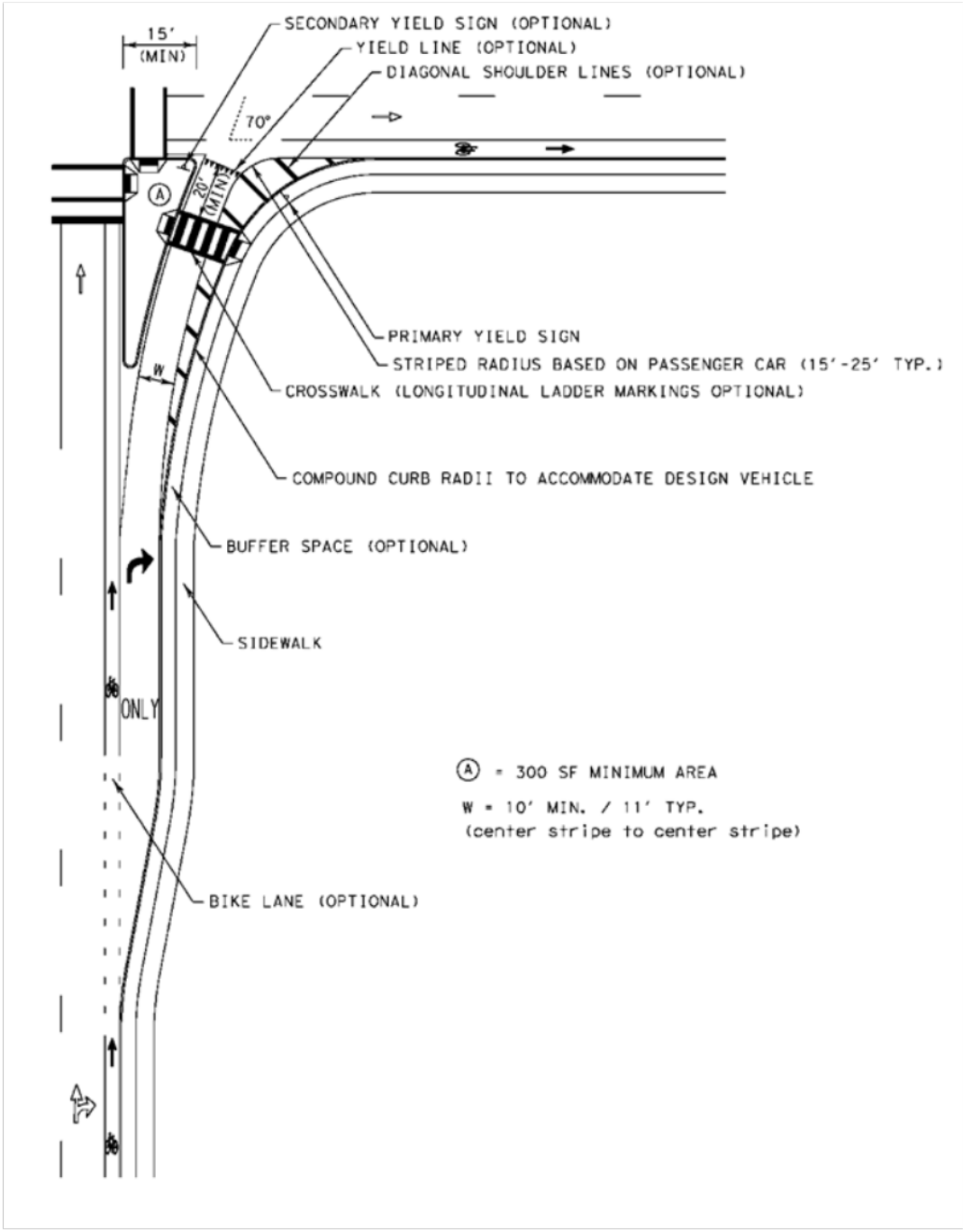


Figure 13-14: Right-Turn Slip Lane Design for Urban Intersections with Deceleration Lane

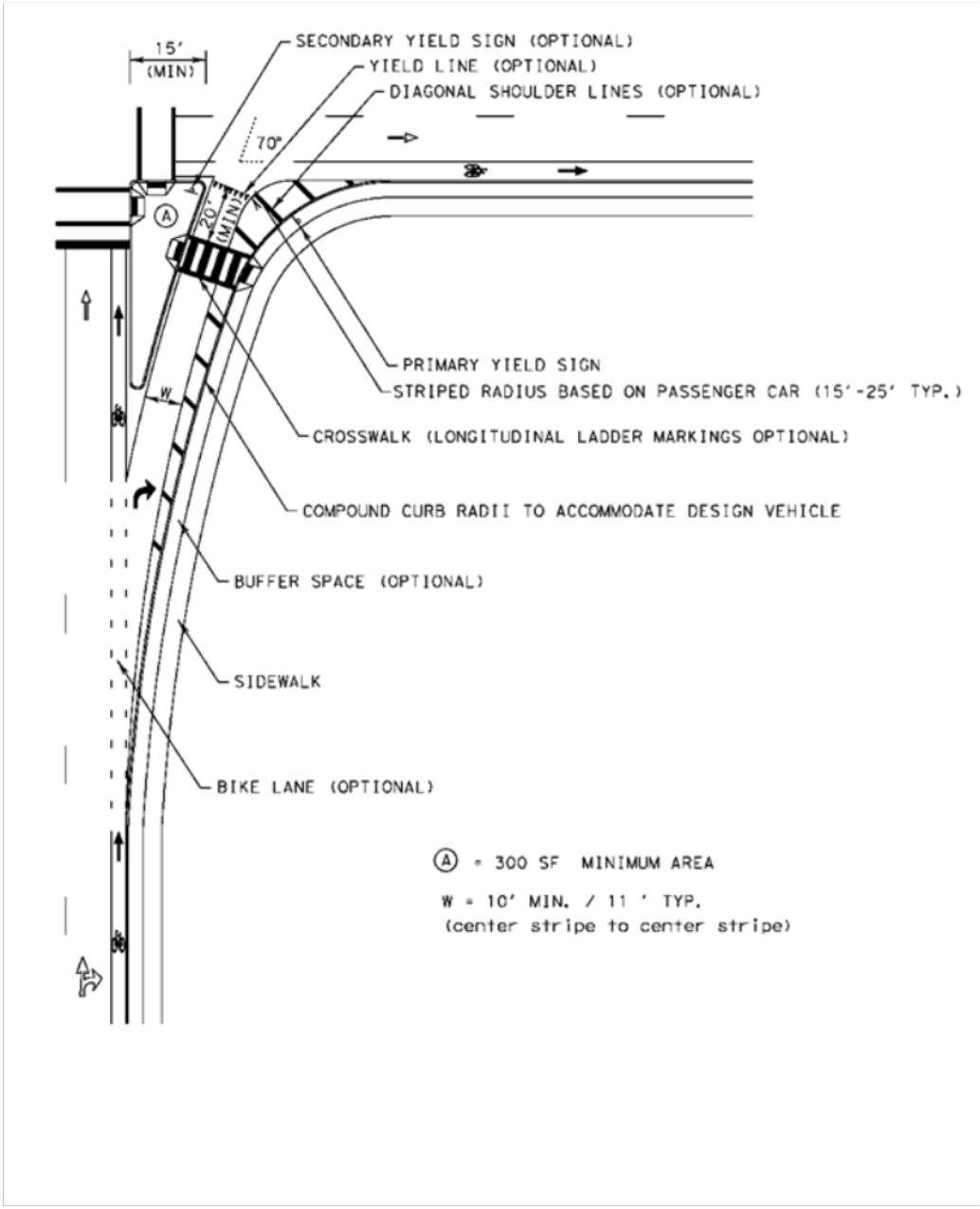


Figure 13-15: Right-Turn Slip Lane Design for Urban Intersections without Deceleration Lane

13.9.2 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.

Angle of Entry - See guidance provided under **Section 13.9.1 - Urban Design, Angle of Entry**.

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 **Figure 13-14** and **Figure 13-15**). See Cross Sectional Elements in **Section 4.10** for striped lane width.

Channelizing Island - See guidance provided under **Section 13.9.1 Urban Design, Channelizing Island**.

Deceleration Lane - See guidance provided under **Section 13.9.1 Urban Design, Deceleration Lane**.

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.

Drainage - See guidance provided under **Section 13.9.1 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.

Lighting - See guidance provided under **Section 13.9.1 Urban Design, Lighting**.

Apparatus and Pole Placement - See guidance provided under **Section 13.9.1 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.

Crosswalk Location - See guidance provided under **Section 13.9.1 Urban Design, Crosswalk Location**. If 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.

Crosswalk Orientation - See guidance provided under **Section 13.9.1 Urban Design, Crosswalk Orientation**.

Crosswalk Markings - See guidance provided under **Section 13.9.1 Urban Design, Crosswalk Markings**.

Signage - See guidance provided under **Section 13.9.1 Urban Design, Signage.**

Bicycle Lane - See guidance provided under **Sectio 13.9.1 Urban Design, Bike Lane**.

13.9.3 Rural Design

If pedestrian activity is expected, see **Section 13.9.2 Suburban Design section**.

If pedestrians are not an issue, the following guidelines may be used:

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.

Radius - For guidance on radius design for different vehicle classes, see **Section 13.10.1**. 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.

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. Refer to Chapter 9 of **AASHTO's A Policy on Geometric Design of Highways and Streets** for design guidance on curb offset and tapering (**Figure 13-11**). For details on island approach treatment and delineation refer to Chapter 3 of the **TMUTCD** and Chapter 9 of **AASHTO's A Policy on Geometric Design of Highways and Streets.**

Deceleration Lane – Refer to Chapter 2 of TxDOT's **AMM** for volume thresholds for installing deceleration lanes. See **Section 4.10.2** in this manual for design guidance for deceleration lanes.

Acceleration Lane – Refer to Chapter 2 of TxDOT's **AMM** 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 motorists 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 4, Section 4.10.2** for design recommendations for acceleration lanes for two-lane and multi-lane rural highways, respectively.

Lighting - Intersections with channelization should be illuminated.

13.10 Additional Intersection Design Considerations

13.10.1 Minimum Turning Radii

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:

- Functional class of intersecting routes and location (urban versus rural);
- Consequences of encroachment into other lanes or the roadside;
- Availability of ROW; and
- Type and frequency of use by large vehicles. TPP may be contacted to obtain volume data for the various design 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 similar software should be used to determine the extent of design vehicle encroachment. A control vehicle should also be considered when designing an intersection. See **Section 4.3.3** for additional information on control vehicles.

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 13-7** shows the minimum edge of pavement designs for right turns to accommodate various design vehicles for turn angles varying from 60 to 120 degrees.

Table 13-7: Minimum Edge of Pavement Designs for Right Turns for Various Design Vehicles for Turn Angle Varying from 60 to 120 Degrees[2]

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
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

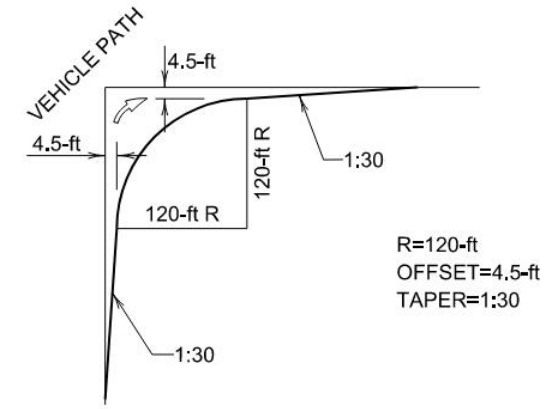
Notes:

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 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.
2. Values are calculated based on the design vehicle's turning path from the outermost lane of the approach roadway to the outermost lane of the crossing roadway.

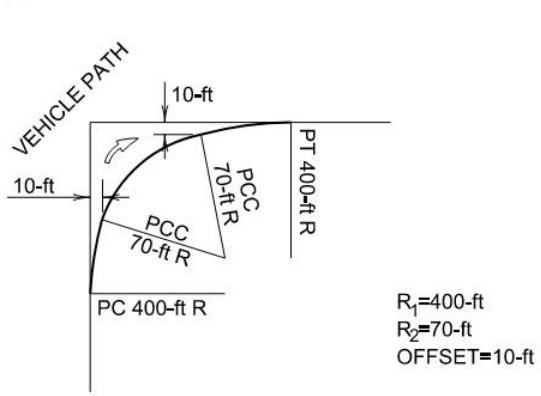
Figure 13-16 shows sample alternatives to simple curvature edge of pavement geometry for a 90-degree turn using a WB-62 design vehicle. Although not shown in this figure, a radius of 100-ft without channelizing island would be necessary to accommodate the wide, off-tracking path of a WB-62 without undesirable 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.

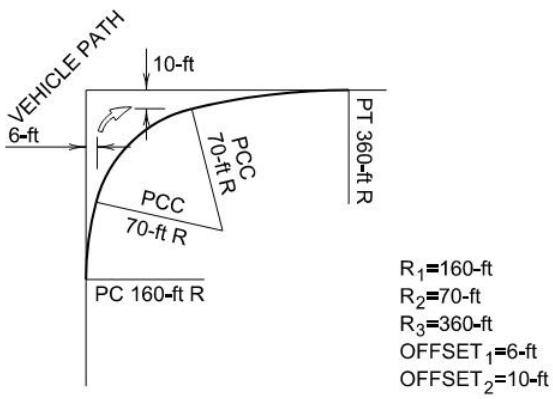
EXAMPLE PAVEMENT EDGE GEOMETRY
WB-62, 90 DEGREE TURN



(A) SIMPLE CURVE RADIUS WITH TAPER



(B) 3-CENTERED COMPOUND CURVE, SYMMETRIC



(C) 3-CENTERED COMPOUND CURVE, ASYMMETRIC

Figure 13-16: 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 ROW 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:

The radii dimensions are for simple curves and should be used as supplemental when designing. The radii should be built to accommodate the design vehicle based on the turning movements.

- 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 only be provided for arterial-arterial urban intersections if frequent use of a WB-62 or larger design vehicle is anticipated. A compound curve or simple curve with taper should be used to minimize an intersection footprint; and
- 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 13-7** and **Figure 13-16** permits 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 are 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 13-7** 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 13-17** 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. See **Section 14.6** for additional information on loon design.

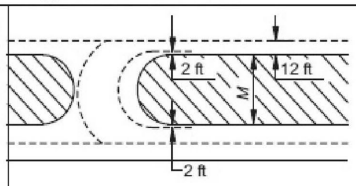
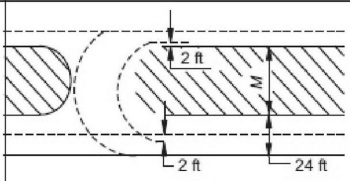
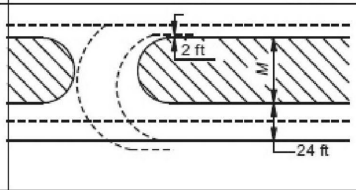
U.S. Customary							
	M—Minimum Width of Median (ft) for Design Vehicle						
	P	WB-40	SU-30	BUS	SU-40	WB-62	WB-67
	Length of Design Vehicle (ft)						
Type of Maneuver	19	50	30	40	40	63	68
Inner Lane to Inner Lane 	30	61	63	63	76	69	69
Inner Lane to Outer Lane 	18	49	51	51	64	57	57
Inner Lane to Shoulder 	8	39	41	41	54	47	47

Figure 13-17: Minimum Median Widths For U-Turn Crossovers

Source: FHWA MUT Informational Guide – Exhibit 7-12

13.11 Railroad-Highway Grade Crossings

13.11.1 Horizontal Alignment

If practical, the roadway should intersect the tracks at a right angle with no nearby intersections or driveways. This layout enhances the driver's view of the crossing and tracks, reduces conflicting vehicular movements from crossroads and driveways, and is preferred for bicyclists and pedestrians.

Highly skewed railroad-highway crossings can cause wheels of bicycles to get caught in the gap between the rail and pavement. To the extent practical, crossings should not be located on either roadway or railroad curves. Roadway curvature inhibits a driver's view of crossing ahead.A driver's attention may be directed toward negotiating the curve rather than looking for a train. Railroad curvature may inhibit a driver's view down the tracks from both a stopped position at the crossing and on the approach to the crossing. Those crossings that are located on both roadway and railroad curves present maintenance challenges and poor rideability for roadway traffic due to conflicting superelevation.

Where roadways that are parallel with main tracks intersect roadways that cross the main tracks, there should be sufficient distance between the tracks and the roadway intersections to enable roadway traffic in all directions to move expeditiously. Where physically restricted areas make it impractical to obtain adequate storage distance between the main track and a roadway intersection, the following should be considered:

- Interconnection of the roadway traffic signals with the grade crossing signals to enable vehicles to clear the grade crossing when a train approaches; and
- Placement of a "Do Not Stop on Tracks" sign on the roadway approach to the grade crossing.

13.11.2 Vertical Alignment

It is desirable from the standpoint of sight distance, rideability, braking, and acceleration distances that the intersection of roadway and railroad be made as level as practical. Vertical curves should be of sufficient length to provide an adequate view of the crossing.

In some instances, the roadway vertical alignment may not meet acceptable geometrics for a given design speed because of restrictive topography or limitations of ROW. To prevent drivers of low-clearance vehicles from becoming caught on the tracks, **the road crossing surface should be level with the top of the rail for a distance of 2-ft from the nearest rail and not more than 3 inches higher or lower than the top of the nearest rail at a point 30-ft from the rail**, as shown in **Figure 13-18**. Vertical curves should be used to traverse from the roadway grade to a level plane at the elevation of the rails. Rails that are superelevated, or a roadway approach section that is not level, need a site-specific analysis for rail clearances.

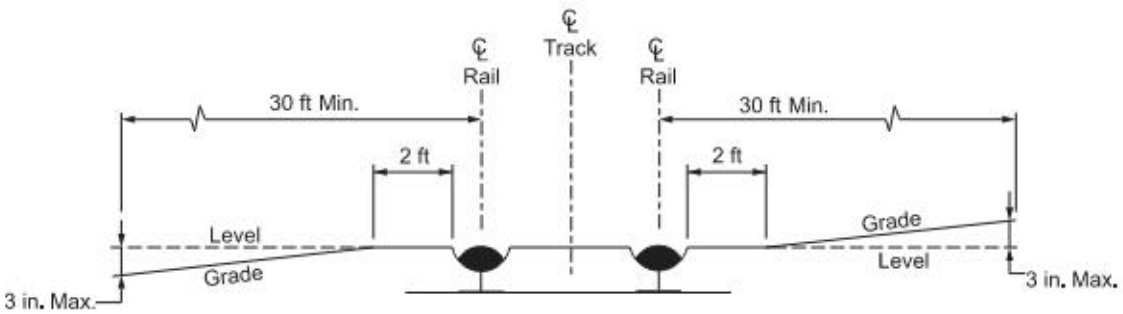


Figure 13-18: Railroad-Highway Grade Crossing

Source: AASHTO A Policy on Geometric Design of Highways and Streets

13.11.3 Crossing Design

The geometric design of railroad-highway grade crossings should be made jointly when determining the warning devices to be used. When only passive warning devices are used, such as signs and pavement markings, the roadway drivers are warned of the crossing location but need to determine for themselves whether there are train movements for which they should stop. On the other hand, when active warning devices such as flashing light signals or automatic gates are used, the driver is given a positive indication of the presence or the approach of a train at the crossing. Many significant variables should be considered in determining the type of warning device to be installed at a railroad-grade crossing. For certain low-volume roadway crossings, where adequate sight distance is not available, additional signing may be needed.

Traffic control devices for railroad-highway grade crossings consist primarily of signs, pavement markings, flashing light signals, and automatic gates. Criteria for the design, placement, installment, and operation of these devices, refer to the **TMUTCD,** as well as the use of various passive warning devices.

Some of the considerations for evaluating the need for active warning devices at a grade crossing include the type of roadway, volume of vehicular traffic, volume of railroad traffic, maximum speed of the railroad trains, permissible speed of vehicular traffic, volume of pedestrian traffic, crash history, sight distance, and geometrics of the crossing. The potential for complete elimination of grade crossings without active traffic control devices (e.g., closing lightly used crossings and installing active devices at other more heavily used crossings) should be given prime consideration.

These guidelines are not all-inclusive. Situations not covered by these guidelines should be evaluated using good engineering judgment. Additional information on railroad-highway grade crossings can be found in the following sources:

- Railroad-Highway Grade Crossing Handbook;

- Railroad-Highway Grade Crossing Surfaces;
- Sight Distance and Approach Speed;
- Traffic Control and Roadway Elements-their Relationship to Highway Safety;
- NCHRP Report 288; and
- Traffic Control Devices and Rail-Highway Crossings.

Numerous index formulas have been developed to assess the relative conflict potential at railroad-highway grade crossings based on various combinations of its characteristics. Although no single formula has universal acceptance, each has its own values in establishing an index. When used with sound engineering judgment, each formula provides a basis for a selection of the type of warning devices to be installed at a given crossing.

The geometric design of a railroad-highway grade crossing involves the elements of alignment, profile, sight distance, and cross-section. The appropriate design may vary with the type of warning device used. Where signs and pavement markings are the only means of warning, the roadways should cross the railroad, at or nearly at, right angles. Even when flashing lights or automatic gates are used, small intersection angles should be avoided. Regardless of the type of control, the roadway gradient should be flat at, and adjacent to, the railroad crossing to permit vehicles to stop when necessary, and then proceed across the tracks without difficulty.

13.11.4 Sight Distance

Sight distance is a primary consideration at crossings without train-activated warning devices. For a complete discussion of sight distance at grade crossings, refer **to TxDOT's Rail Highway Operations Manual - Railroad Crossing Surfaces** and **NCHRP Report 288**. Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for sight distance calculations.

Chapter 14
Alternative Intersections and Interchanges

14.1 Introduction

An alternative intersection or interchange is one where traffic movement is re-routed from a conventional intersection form to remove major conflict points. To help reduce intersection related crashes, and as traffic demand increases, the use of alternative intersections, such as roundabouts, diverging diamond interchanges, restricted crossing U-turns, and continuous flow intersections, is becoming more prevalent within the United States. The target speed should be lowered as vehicles approach and go through roundabouts and some other alternative intersection types to achieve proper functioning and the resulting safety benefits.

Some of the advantages of these types of intersections or interchanges include the following:

- A general decrease in conflict points and crashes, and particularly a reduction in higher-severity crashes;
- An improvement in traffic flow by eliminating, relocating, or modifying conflict points;
- Improved signal phasing and operations;
- A decrease in congestion and a reduction in traffic bottlenecks;
- Improvement of intersection delay, Level of Service (LOS), travel time, and vehicle throughput;
- Improved ability to maintain existing bridge structures; and
- A possible reduction in the amount of ROW required for new projects.

This chapter discusses the following sections and subsections:

14.1 Introduction

14.2 Pedestrian Considerations for Alternative Intersections

14.3 Intersection Analysis/Intersection Control Evaluation

14.4 Roundabouts

- 14.4.1 Overview;
- 14.4.2 Goals of a Roundabout Design;
- 14.4.3 Design Process and Design Context;
- 14.4.4 Design Parameters;
- 14.4.5 Performance Checks;
- 14.4.6 Special Considerations; and
- 14.4.7 Preliminary and Final Design.

14.5 Continuous Green T-Intersection (CGT)

- 14.5.1 Overview;
- 14.5.2 Design Considerations;
- 14.5.3 Sight Distance;
- 14.5.4 Access Management; and

- 14.5.5 Pedestrian and Bicyclist Considerations.

14.6 Restricted Crossing U-Turn Intersection (RCUT)

- 14.6.1 Overview;
- 14.6.2 Design Considerations;
- 14.6.3 Intersection Angle;
- 14.6.4 Pedestrian and Bicyclist Considerations;
- 14.6.5 Access Management;
- 14.6.6 Safety;
- 14.6.7 Operations; and
- 14.6.8 Traffic Control Devices.

14.7 Displaced Left-Turn Intersection (DLT)

- 14.7.1 Overview;
- 14.7.2 Design Considerations;
- 14.7.3 Pedestrian and Bicyclist Considerations;
- 14.7.4 Access Management;
- 14.7.5 Safety;
- 14.7.6 Operations; and
- 14.7.7 Traffic Control Devices.

14.8 Median U-Turn Intersection (MUT)

- 14.8.1 Overview;
- 14.8.2 Design Considerations;
- 14.8.3 Pedestrian and Bicyclist Considerations;
- 14.8.4 Access Management;
- 14.8.5 Safety;
- 14.8.6 Operations; and
- 14.8.7 Traffic Control Devices.

14.9 Quadrant Roadway Intersections

14.10 Diverging Diamond Interchange (DDI)

- 14.10.1 Overview;
- 14.10.2 Design Considerations;
- 14.10.3 Sight Distance;
- 14.10.4 Horizontal Alignment Alternatives;
- 14.10.5 Auxiliary Lanes;
- 14.10.6 Ramp Terminals;
- 14.10.7 Pedestrian and Bicyclist Considerations;
- 14.10.8 Access Management;
- 14.10.9 Safety;
- 14.10.10 Operations;
- 14.10.11 Traffic Control Devices; and
- 14.10.12 Illumination.

14.11 Contraflow Left Interchange (CLI)

14.12 Single-Point Urban Interchange

14.13 Roundabout Interchange

14.14 Partial Cloverleaf Interchange

Refer to the **TSAP** for additional discussion on the different types of alternative intersections and alternative interchanges including:

Alternative Intersections

- Roundabout;
- Continuous green-T (CGT);
- Restricted Crossing U-turn (RCUT);
- Displaced Left-turn Intersection (DLT);
- Median U-turn (MUT); and
- Quadrant Roadway.

Alternative Interchanges

- Diverging Diamond Interchange (DDI);
- Contraflow Left Interchange;
- Single-Point Urban Interchange (SPUI);

- Roundabout; and
- Partial Cloverleaf.

The public should be involved in the selection process when determining the appropriate alternative interchange or intersection. Refer to the National guidance documents referenced in each respective section of this chapter for additional guidance on public involvement best practices.

14.2 Pedestrian Considerations for Alternative Intersections

While pedestrian considerations for alternative intersections require close examination, general considerations for alternative intersection design treatments include:

Ensuring sight distance for pedestrians and motorists and minimizing obstructions. This is particularly important as vehicles and pedestrians are often not approaching one another at right angles.

- Providing wayfinding for navigation;
- Safe and comfortable traversal of the intersection;
- Reducing radii values to decrease vehicle speed;
- Compliance with the **Americans with Disabilities Act of 1990 (ADA)** and **Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-way (PROWAG)** requirements; and
- Reducing the number of multiple conflict-point locations for crossings.

For more information on non-standard roadway treatments, refer to **NCHRP Report 948 Guide for Pedestrian and Bicycle Safety at Alternative and Other Intersections and Interchanges**.

14.3 Intersection Analysis/Intersection Control Evaluation (ICE)

Intersection Control Evaluation (ICE) 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. Several tools and documents are available to aid in assessing the eligibility of an intersection for alternative intersection consideration. Refer to **TSAP** for traffic and safety evaluation processes and tools. The following TSAP chapters should be considered:

- TSAP, Chapter 10 – Intersection Analysis;
- TSAP, Chapter 11 – Interchange Analysis; and
- TSAP, Chapter 12 – Roundabout Analysis.

14.4 Roundabouts

This section discusses the features and design criteria for roundabouts

Information in Section 14.4 of the RDM takes precedence over NCHRP Report 1043 where a deviation exists within the guidelines.

14.4.1 Overview

A roundabout, in general, is a circular intersection in which traffic travels counterclockwise around a central island and entering traffic yields to circulating traffic. Geometric features slow traffic at the intersection. There are four classifications of roundabouts: mini-roundabouts, compact roundabouts, single-lane roundabouts, and multilane roundabouts (see **Figure 14-1**). Mini- and compact roundabouts are two types of reduced-diameter roundabouts with traversable features. While mini-roundabouts have a fully traversable center island and may have traversable splitter islands, compact roundabouts have a combination of traversable elements but do not provide a fully traversable center island. See **Table 14-1** for the varying typical diameters for the four classifications of roundabouts.

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 1043 Guide for Roundabouts (2023)** as the primary source for roundabout design guidelines. The information contained in this section of the **RDM** is considered a companion guide to **NCHRP Report 1043**.



Mini-Roundabout



Compact Roundabout



Single-lane Roundabout



Multilane Roundabout

Figure 14-1: Four Classifications of Roundabouts

14.4.2 Goals of a Roundabout Design

Several overarching principles guide the development of all roundabout designs:

- Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume balance, and lane continuity to ensure that the roundabout operates at an appropriate level of service;
- Design to accommodate the largest anticipated vehicle to enter, circulate and depart efficiently while also providing adequate speed control for passenger vehicles;
- The circulating roadway should be wide enough to accommodate the local jurisdiction largest vehicle (fire truck or city bus), with a truck apron provided around the central island for overtracking of the rear axles on larger vehicles;
- A central island that is clearly visible from the approach sight distance at the

road operating speed in advance of the roundabout. Landscaping or hardscaping within the central island can be effective in restricting sight distance to the minimum requirements while creating a terminal vista on the approach to improve visibility of the central island;

- Drivers must be able to see to the left early enough to safely enter the roundabout. However, excessive intersection sight distance may lead to higher vehicle speeds that reduce the safety of the intersection for all road users (motorists, bicyclists, pedestrians);
- Design such that the driving task is as simple as possible, avoiding the use of spiraled lanes unless warranted by traffic (i.e., dedicated left-turns and other reasons cited in **NCHRP Report 1043**, Section 10.7.7); and
- Site-specific design to meet the needs of pedestrians and cyclists.

14.4.3 Design Process and Design Context

Varying characteristics will determine the selection of a roundabout's size, lane configuration, and feasibility. Refer to **NCHRP Report 1043**, Chapter 3, Chapter 8, and Chapter 9 for roundabout performance-based planning and design information and roundabout design principles and objectives.

14.4.3.1 Roundabout Implementation Challenges

It is beneficial to confirm that a roundabout is a feasible solution with respect to site-specific constraints early in the planning process. Advance coordination with adjacent project stakeholders (railroad representatives, neighboring jurisdictions, or local elected officials), review of boundary survey information, and a site visit to observe existing conditions are important steps in validating a potential roundabout installation early in the analysis and design process. Common site-specific constraints consist of:

- Adjacent features that may lead to queue spillback into the roundabout (overcapacity traffic signal, freeway entrance ramp, overcapacity driveway, etc.);
- Unfavorable topography that results in limited visibility, excessive cross slopes in the circulatory roadway, or significantly complicates construction;
- Where the ratio of major roadway traffic to minor roadway traffic exceeds 9:1. Unacceptable delay may result along the minor roadway. Detailed traffic analysis in this situation is recommended. Refer to Chapter 12 of the **TSAP** for additional guidance;
- Significant pedestrian and bicycle movements at a near- or over-capacity roundabout;
- Within a coordinated traffic signal network. A roundabout may disrupt progression and have a negative impact on the efficiency of a signalized corridor.

If possible, converting a corridor to all roundabouts may provide operational efficiencies. Refer to Chapter 9 the **TSAP** for more information regarding segment analysis;

- Roadways with reversible lanes for alternating peak period traffic flows;
- At or adjacent to an at-grade railroad crossing; and
- Environmental factors. Proximity of historical sites, ROW constraints, or other environmental constraints adjacent to an intersection.

A performance-based mindset affords the creation of functional designs with the most flexibility. Due consideration of the intersection--specific physical and traffic conditions throughout the life cycle of a subject intersection will be one of the key contributors to the success of a roundabout installation. TxDOT has several tools and resources available for the collection and analysis of traffic data to prepare an operations analysis, including the data sources outlined in the **TSAP**, Chapters 10, 11 and 12.

14.4.3.2 Identify Design Considerations

Within the planning and conceptual process of a proposed roundabout, a documentation of roundabout parameters is encouraged so the engineer-of-record and corresponding reviewing agencies can agree to the parameters selected for a roundabout's design. Determination of the following parameters after an operational analysis and prior to the concept roundabout sketch can reduce re-sketching and future requests for changes to a roundabout design:

- Inscribed circle diameter (ICD);
- The proposed surface pavement section (concrete or asphalt);
- Mountable/non-mountable curb types;
- Gutter pan dimensions and slopes, if applicable;
- Minimum and maximum performance check values;
- Truck traffic accommodation (straddled lane design vs non-straddled), formerly known as Case 1, Case 2, and Case 3 (refer to **NCHRP Report 1043**, Section 9.7.1);
- Traditional lane geometries, spiraled lanes, buffered lanes, or turbo roundabout design;
- Pedestrian and bicycle facilities, including buffer width between the back of curb;
- The inclusion of bike ramps;
- Proper illumination on the perimeter roadway and within the center landscaped island;
- Maximum cross slopes in the circulatory;
- Adjacent driveway accessibility; and
- If multilane, the use of a traffic control signal with a pedestrian signal head, a pedestrian hybrid beacon (PHB), a pedestrian actuated rectangular rapid flashing beacon (RRFB), or a raised crossing as a crosswalk treatment.

The performance-based geometric design of a roundabout is an iterative process, as outlined in **NCHRP Report 1043**, Exhibit 9.1. It may take multiple revisions to the size of the ICD

and the location of the ICD before an optimized solution is arrived at for the subject intersection.

Considerations to the ICD size and placement of a roundabout consist of:

- Traffic patterns and volumes;
- Existing and/or proposed ROW boundaries;
- Connecting roadway alignments and centerlines;
- The footprint of the existing intersection;
- Circular versus elliptical design;
- The effect the diameter has on geometric speed control and resulting circulating speeds;
- If intersecting roadway legs are skewed, or normal to the intersection;
- The design vehicle and check vehicle(s) – see **Table 14-1** for a list of typical design and check vehicles and **Section 14.4.5.4** for an explanation of design and check vehicles;
- Oversize or overweight vehicles (OSOW);
- Environmental considerations; and
- Grade considerations.

Table 14-1: Common ICD Ranges

Source: Adapted from NCHRP Report 1043

Roundabout Configuration	Typical AASHTO Design Vehicle[1]	Typical AASHTO Check Vehicle	Common ICD Ranges
Mini-roundabout	Local jurisdiction largest apparatus (fire truck or city bus) [2]	WB-62TX	70-ft to 90-ft
Compact Roundabout (non-traversable central island)	Local jurisdiction largest apparatus (fire truck or city bus) [2]	WB-62TX	90-ft to 110-ft
Single-lane roundabout (non-traversable central island)	Local jurisdiction largest apparatus (fire truck or city bus) [2]	WB-62TX, WB-67 or OSOW	110-ft to 150-ft
Multilane roundabout (2 lanes circulating)[3]	Local jurisdiction largest apparatus (fire truck or city bus) [2]	WB-62TX, WB-67 or OSOW	140-ft to 200-ft
Multilane roundabout (3 lanes circulating)[3]	Local jurisdiction largest apparatus (fire truck or city bus) [2]	WB-62TX, WB-67 or OSOW	180-ft to 220-ft

Notes:

1. Assumes 90-degree angles between entries and no more than four legs. List of possible design vehicles is not comprehensive.
2. Certain Design Vehicles must be able to navigate the roundabout without the use of the truck apron or external truck apron.
3. The Common ICD Ranges shown for multilane roundabouts represents typical ranges to be used for straddle-lane design. For stay-in-lane design, or OSOW accommodation, slightly larger diameters may be necessary.

Mini-roundabouts are not common on State routes, due to the higher speed context and presence of added large trucks. Locating mini-roundabouts in suburban areas with moderate speed context is feasible with added approach

treatment of reverse curves and longer splitter islands.

Compact and single-lane roundabout design is commonly influenced by design vehicle swept path requirements. Rural design requires provision of a visibility package of approach geometry (frequently using reverse curves), signs, illumination, and markings.

14.4.4 Design Parameters

The following design parameters and their associated values shall be documented and agreed upon by all relevant parties prior to a conceptual or schematic roundabout design. Advance determination of these parameters and values is beneficial to reduce the potential for future re-work of the roundabout design.

- Inscribed Circle Diameter, face of curb to face of curb (FOC-FOC);
- Circular ROW Footprint (ICD plus sidewalk and buffer strip, ft.);
- Design and Check Vehicle(s), per movement (i.e. If a larger vehicle is known to only make a right-turn at a given intersection, the remaining turning movements may be assigned a separate check vehicle) (see **Section 14.4.5.4**);
- Fastest Path Geometric Entry Speed, R1 (method: **NCHRP Report 1043**, Section 9.4);
- Fastest Path Entry/Circulating Speed Differential (V1 to V2) (see **Section 14.4.5.2**);
- Minimum Circulatory Cross Slope (%);
- Maximum Longitudinal Grade (% approach or circulating);
- Maximum Cross Slope (low side if circulatory roadway is tipped [i.e. tilted plane design]);
- Volume Range (sum of entering and conflicting volumes);
- Splitter Island Length (length will vary depending on if posted regulatory speed is less than or equal to 35 mph, or greater than 35 mph);
- Entry Curb Radius (near the yield line, ft.);
- Exit Curb Radius (ft.);
- View Angle (angle in degrees from a straight-ahead driver's view) (see **Section 14.4.5.6**);
- Entry Width (see **Section 14.4.4.1**);
- Exit Width (see **Section 14.4.4.1**);
- Circulatory Roadway Width, FOC-FOC (ft.);
- Inner Circulatory Lane Width (ft.);
- Outer Circulatory Lane Width (ft.);
- Truck Apron Width (ft.) (see **Section 14.4.4.4**);
- Exit tangency (for path overlap check). Refer to **NCHRP Report 1043**, Exhibit A.18, and see **RDM** Section 14.4.5.6;
- Curb Offsets (ft.);
- Buffer Width (FOC to Sidewalk, ft.);
- Crosswalk Pedestrian Refuge Width (ft.);
- Crossing Accommodation (Traffic Control Devices and/or the use of raised crosswalks); and
- Maintenance of Traffic during construction (highway and/or lane closures).

NCHRP Report 1043, Section 10.7 provides justification for added emphasis of the following in multilane roundabout design:

- Geometric speed control;
- Avoidance of complicated lane configurations and use of single-lane entries and exits where the capacity trade-off is negligible;
- Development of natural parallel entry and exit paths;
- Provision of optimal approach lane markings; and
- Overhead signs for lane designation.

Figure 14-2 to **Figure 14-5** highlight ranges of common design considerations for various geometric parameters of roundabouts for the four different classifications.

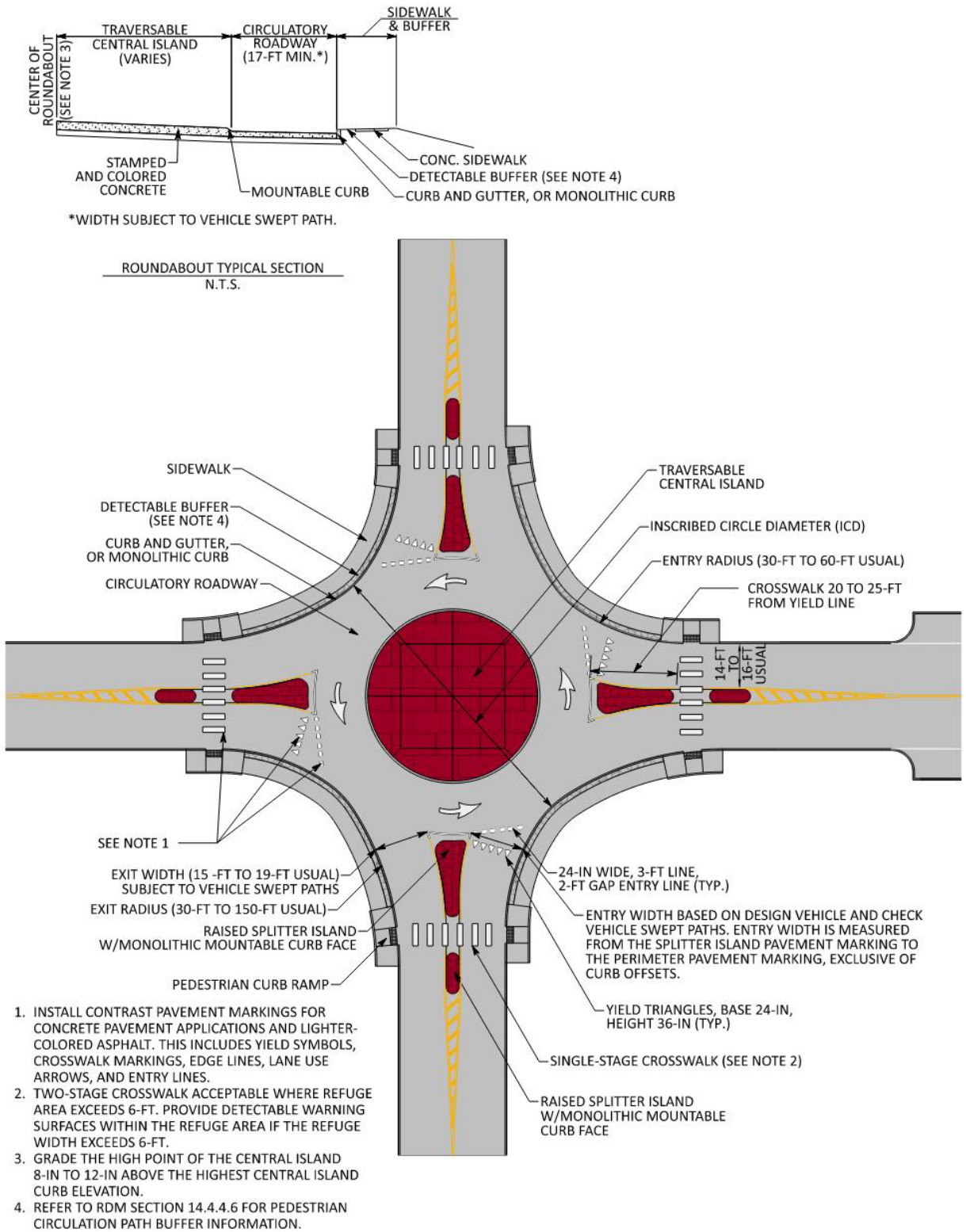


Figure 14-2: Mini-Roundabout Design Parameters

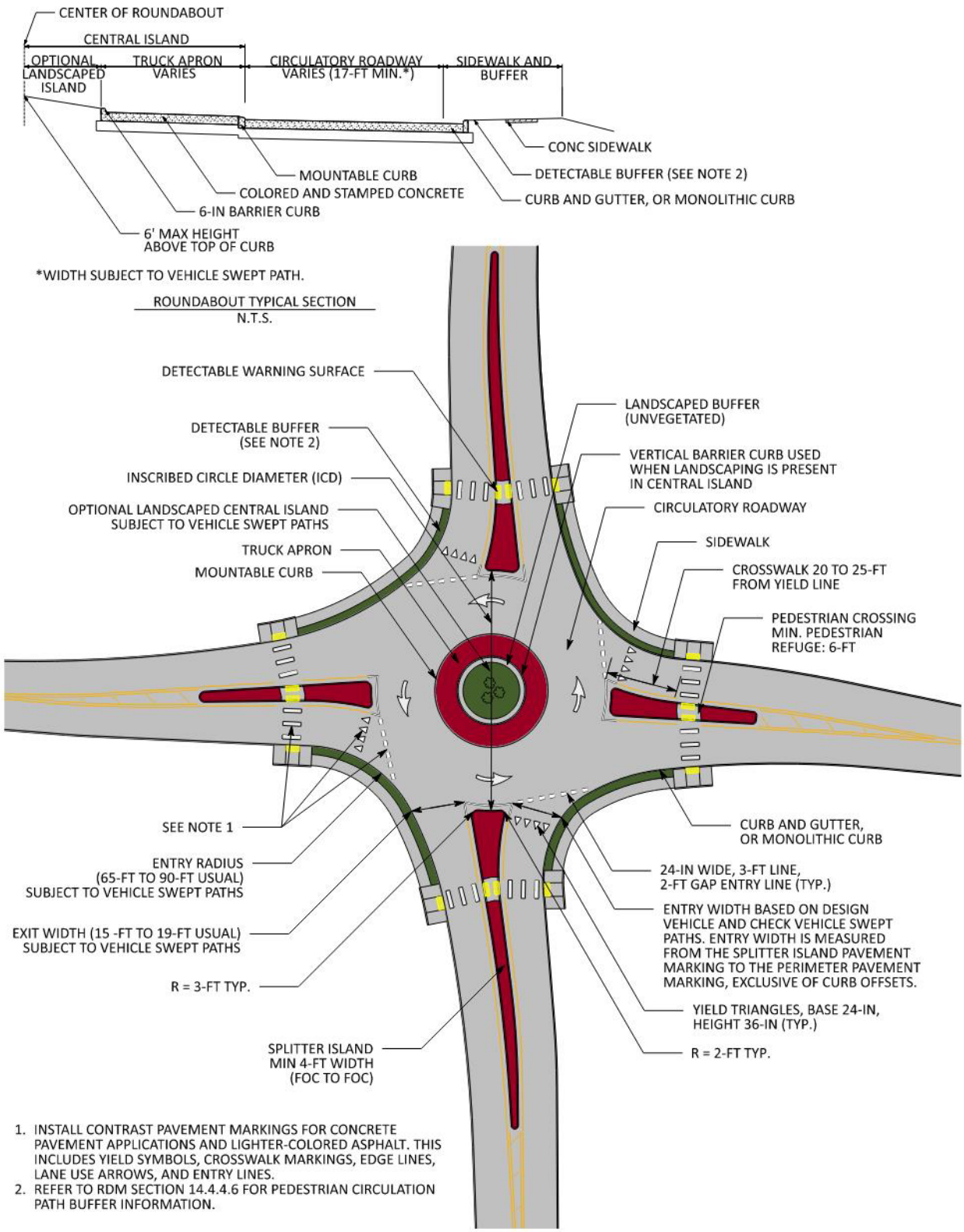


Figure 14-3: Compact Roundabout Design Parameters

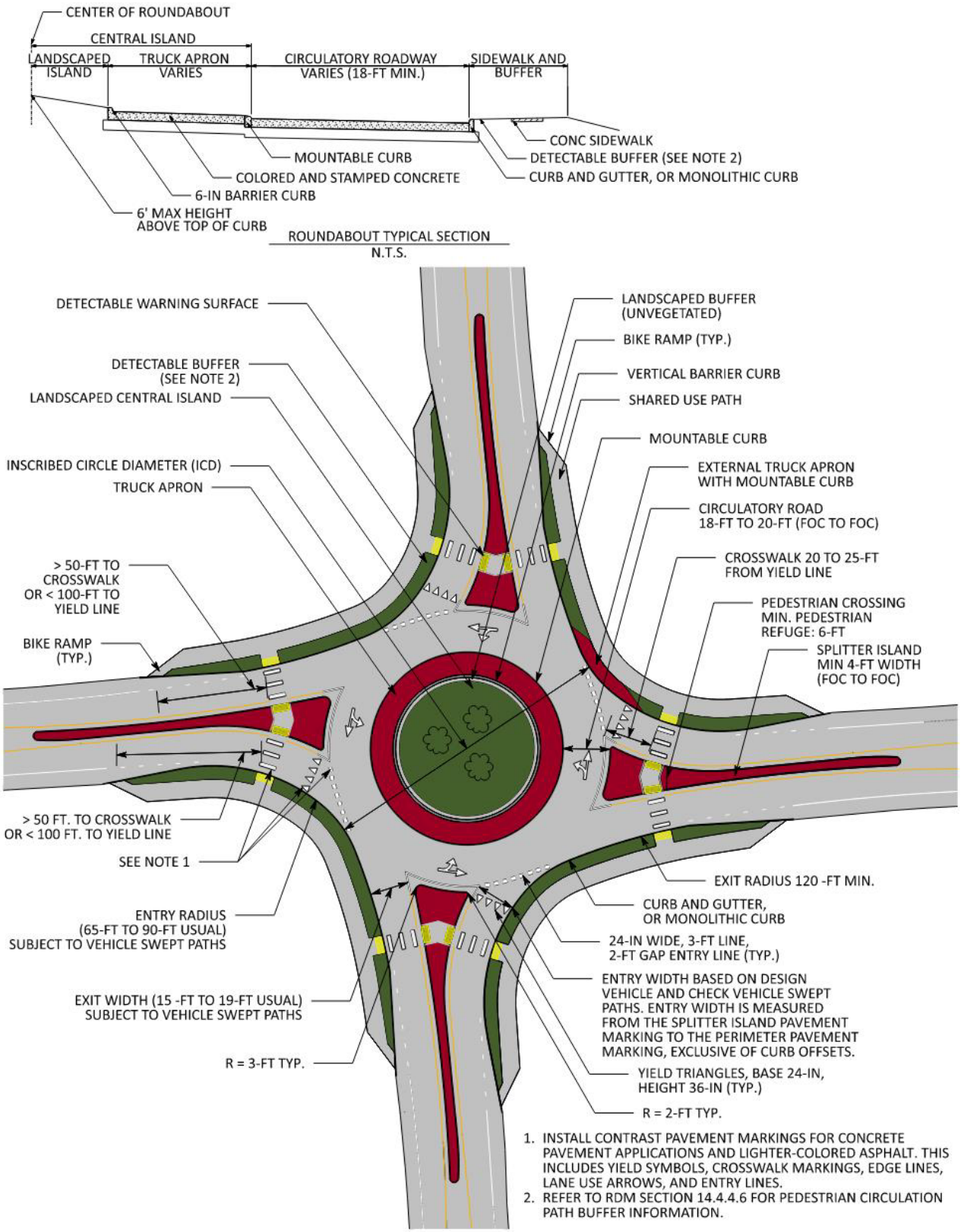


Figure 14-4: Single-Lane Roundabout Design Parameters

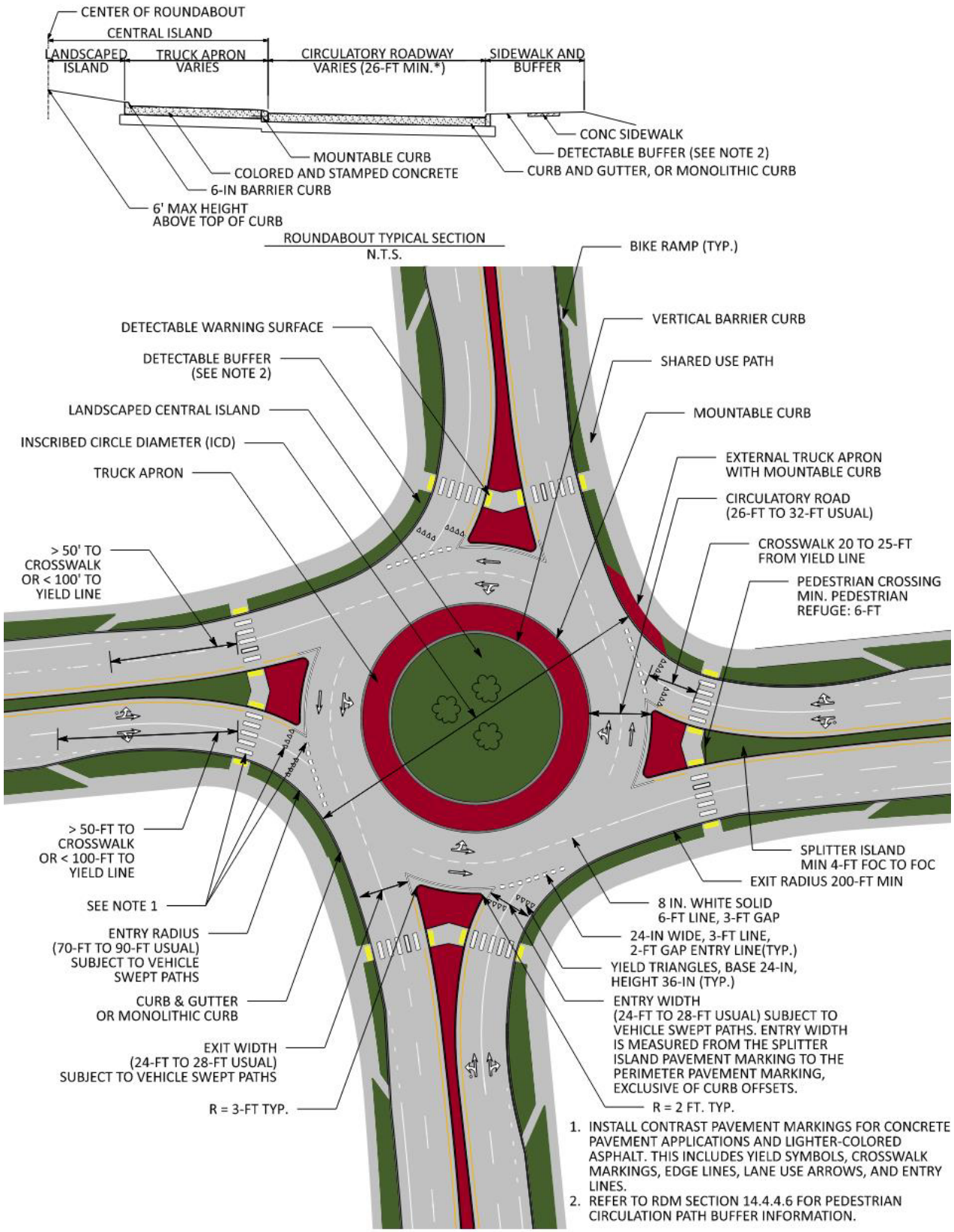


Figure 14-5: Multilane Roundabout Design Parameters

14.4.4.1 Entry and Exit Geometry

NCHRP Report 1043, Section 10.6.4 and Section 10.6.5 detail entry and exit design for roundabouts.

The entry width is measured from the center of the pavement marking that is adjacent to the splitter island to the center of the pavement marking adjacent to the exterior curb, or if a gutter section is present, the width is measured to the edge of the traveled way.

The design vehicle and check vehicle must be able to navigate the entry of a single-lane roundabout with a one-foot shy distance from the wheel path to the face of curb or edge of traveled way. When a WB-62TX is to be accommodated at a single-lane roundabout as the check vehicle, a wider entry width may be required due to the swept path of the rear tandem axle tires relative to the cab's front tires.

At multilane roundabouts the entry width will depend on the design case. If stay-in-lane design is employed, then the resulting entry width will be greater than the typical straddle lane design entry width to accommodate the check vehicle in-lane while providing the necessary gore width between adjacent entry lanes. Individual lane widths for two-lane entries should remain in the 12-ft to 13-ft range. Maintaining trucks in-lane has safety consequences for other users including longer pedestrian crossings and higher in-lane speeds. Generally, promoting in-lane design for the design vehicle should be restricted to interchanges.

Exit design will rely heavily on single-lane or multilane designation, subsequent performance checks (fastest path and check vehicle), and the exit alignment that connects the roundabout to the adjacent roadway. Straighter exit paths provide ideal sight of the exit crosswalks, but exit speeds can increase, which places a greater emphasis on controlling entry speed.

14.4.4.2 Circulatory Roadway

For single-lane roundabouts, an 18-ft to 20-ft (face of curb to face of curb) circulatory roadway width is typical. For two-lane straddle-lane roundabouts, circulatory width is commonly in the range of 26-ft to 32-ft. For two-lane stay-in-lane design for trucks ("buffered lane design"), the circulatory width may be wider to accommodate the check vehicle being analyzed for both lanes in combination with the gore area separating the two circulatory lanes. Circulating lane widths should split the remaining circulatory roadway evenly.

14.4.4.3 Splitter Islands

NCHRP Report 1043, Section 10.6.1 and 10.14.2 discusses splitter island types and lengths. Splitter island design discussion can be found throughout **NCHRP Report 1043** due to a splitter island's influence on pedestrian and bicycle accommodation, speed control, deterring wrong-way driving, and space allocation for traffic control devices. Splitter islands may be bounded by full-height barrier curb, mountable curb, or a flush fully traversable painted area. The context of the intersection should dictate the elements

chosen for the splitter island. The minimum length of a splitter island for low-speed urban design is 50-ft. See **Section 14.4.6.3** for additional guidance for splitter islands at high-speed context roundabouts.

14.4.4.4 Truck Apron

NCHRP Report 1043, Section 10.5.2 discusses truck aprons. Guidance for truck apron sizing, materials, and even the relationship that truck aprons provide to maintenance can be found within **NCHRP Report 1043**. It is important for the truck apron to have contrasting color and texture from the circulating pavement within the roundabout. This contrast allows drivers, especially during nighttime conditions, an increased ability to identify the limits of the circulatory roadway.

Consider the installation of yellow raised pavement markers spaced at 4-ft to 6-ft along the perimeter of the truck apron curb for mini-roundabouts to enhance nighttime visibility of the central island.

14.4.4.5 External Truck Apron

When performing the check vehicle performance checks, the usage of an external truck apron (a.k.a. "perimeter truck apron") may be required to accommodate the wheel path of the check vehicle. The horizontal geometry of external truck aprons relies on the designer's understanding of how to navigate the check vehicle through the respective movement (typically the right-turn but may also be required for through movements**). NCHRP Report 1043**, Section 11.6 discusses the use of external truck aprons. The use of external truck aprons should be avoided, if possible, by incremental modifications to the approach and entry geometry, entry radius, exit geometry, and exit radius, such that the modifications still result in a design that is validated by the standard performance checks.

14.4.4.6 Pedestrian and Shared-Use Crossings

NCHRP Report 1043, Section 10.4.7 highlights the three basic types of pedestrian and bicycle crossings at roundabouts, angled, straight, and z-crossing. A best practice at roundabouts, especially where bike ramps are implemented, is the use of tactical directional indicators (TDIs). Refer to **NCHRP Report 1043**, Section 10.4.3 for information regarding TDIs. Key considerations for pedestrian and bicycle crossing design at roundabouts consist of:

- Adequate space for pedestrian and bicycle refuge within the splitter islands;
- Bicycles that haul child trailers or cargo trailers may require additional width beyond the minimum prescribed six feet from face-of-curb to face-of-curb;
- Single-stage crossings where the refuge width in the splitter island is less than 6-ft and detectable warning surfaces (DWS) are omitted, versus a typical two-stage crossing where pedestrian refuge is accounted for between opposing directions of travel within the splitter island;
- Positioning and alignment of TDIs, when applicable; and
- 20-ft to 25-ft distance from the entry line adjacent to the nearest point of the

designated crossing. Refer to **NCHRP Report 1043, Exhibit 10.28.**

Pedestrian circulation paths at roundabouts between crosswalks shall be separated from the curb with landscaping or other nonprepared detectable surface with a minimum width of 24 inches. Exception: If a curb-attached pedestrian circulation path is provided, it shall have a continuous and detectable vertical edge treatment along the street side of the path, from crosswalk to crosswalk. The bottom edge of the vertical edge treatment shall be 15 inches maximum above the pedestrian circulation path. Refer to Section R306.4.1.1. and R306.4.1.2. of the **Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way, August 2023 Federal Register Rules Update**, or latest version.

14.4.5 Performance Checks

The validation of a roundabout's horizontal geometry is one of the most important steps in the design process. Roundabout performance checks should be completed prior to any vertical design or supplemental detailing of a roundabout. **NCHRP Report 1043**, Chapter 9 highlights the various current industry standard performance checks associated with fastest path speeds, sight distance, view angles, vehicle path alignments, design and check vehicles, and bicycle and pedestrian wayfinding and crossing assessment. Appendix A of **NCHRP Report 1043** outlines performance check techniques. Performance checks completed in CAD software are required to remain a part of the project's design files throughout the project life cycle, including post-construction, in case an audit of the roundabout occurs after the roundabout is constructed.

14.4.5.1 Documentation Requirements

A roundabout design performance checks package is required. The documentation should be compiled in a series of design check diagrams as follows:

Series 1 – Overview and Horizontal Design - A dimensioned general layout drawing showing:

- Inscribed circle diameter, circulatory roadway width, entry and exit widths, splitter island lengths, and truck apron widths;
- Pedestrian crossing locations/width, composition, raised/flush, markings, when applicable;
- Bicycle lane/approach & termination point, when applicable;
- A typical cross-section of the roundabout's central island (mounded), roadway and truck apron compositions and integration, widths, slopes, and curb standards;
- Location of nearest entrances from the inscribed circle diameter and the nature of the land use; and
- Proximity of roundabout to traffic signal or other intersections.

Series 2 - Geometric design speed ('fast path') diagrams for each approach, entry, circulating, and exit are required. Roundabouts must be designed to promote geometric speed control per theoretical fastest path speeds in accordance with **NCHRP Report 1043**, Section

9.4 and **Figure 14-6**. Left offset of the approach alignment increases the space for the approach curve to be well-developed for entry speed reduction. Geometric entry speeds, measured according to **NCHRP Report 1043**, must be less than 25 mph along the fastest through movement path for single-lane roundabouts and less than 28 mph for the fastest through path for multilane roundabout entries. Approach roadway design speeds and fastest theoretical paths are to be shown. Fast path radii and the range of corresponding speeds based on a (+)2.0% cross slope and (-)2.0% cross slope are to be annotated on the design checks submittal Series 2 diagrams.

Series 3 - Truck swept-path diagrams are to be developed using AutoTURN®, Vehicle Tracking, or similar software, with depiction of the design and check vehicle(s) for all of the anticipated turning movements. Design and check vehicles will validate the width of lanes and the circulatory roadway, e.g., S-BUS-40 or CITY-BUS. The check vehicle shall validate the width of the central island truck apron and entry and exit widths using a 1-ft shy distance from face of curb (FOC) to the vehicle wheels. The truck cab of a design and check vehicle will typically stay on the circulatory roadway when running swept-path analysis with all tires remaining at least 1-ft from the face of curb.

Series 4 – Stopping sight, intersection sight and view angle diagrams accounting for both vertical approach grades and horizontal alignments. Sight distances must account for driver eye height and height of objects. This is for the purpose of user safety and to establish limits to planting scheme/landscaping for the mounded central island and splitter islands. Refer to **NCHRP Report 1043**, Section 9.5.

Series 5 – Ancillary elements diagrams include light pole locations and traffic control devices such as signing and pavement markings (refer to the **TMUTCD**). These elements are generally developed after schematic design; however, with multilane roundabouts, it is necessary to design markings at the concept or schematic stage to validate the use of exclusive lanes and spiraled geometry.

14.4.5.2 Geometric Speed Control (Fastest Path Speed Measurement)

NCHRP Report 1043, Section 9.4.1 and Appendix A.1 covers the assessment of speed control for a roundabout's horizontal geometry. Section 9.4.2 of **NCHRP Report 1043** outlines the process for developing fastest path alignments using the spline technique. Key considerations for fastest path speed assessment for roundabouts consist of:

- Correct placement of node points to develop the associated spline;
- Using Control Points (MicroStation) as a method for creating the spline within CAD software;
- Using the correct offset dimension for the center of vehicle location relative to curbs or pavement markings;
- Producing a spline that is located within 5.0-ft to 5.1-ft of the controlling offset feature (face of curb or pavement marking), or 3.0-ft to 3.1-ft where opposing traffic adjacent to the roadway

centerline is separated by 5-ft or greater;

- Avoiding the addition of node points beyond the typical nine nodes for a through movement, seven nodes for a right-turn movement, and nine nodes for a left-turn movement;
- Adjusting node points along the 165-ft, 215-ft, or 265-ft offset arcs from the ICD when creating a spline that does not cross the minimum 5.0-ft offset along the approach or exit geometry of the roundabout;
- Maintaining less than a maximum differential between V1 to V2 and V1 to V4 of 10 mph;
- Use of a maximum allowable fastest path entry speed, V1, to a value of 28 mph for a multilane entry design;
- Use a consistent 70-ft arc length along the spline when calculating the corresponding R-value; and
- Comparing R2 values on opposite sides of the central island. Unbalanced R2 values can be an indicator that the ICD chosen for the roundabout design may not be optimized and should be shifted in the direction of the higher R2 value to provide enhanced and balanced speed control.

Figure 14-6 provides a graphical representation of the corresponding theoretical maximum fastest path speed alignments for a left-turn (R4/V4), through movement (R1/V1, R2/V2, R3/V3), and a right-turn (R5/V5).

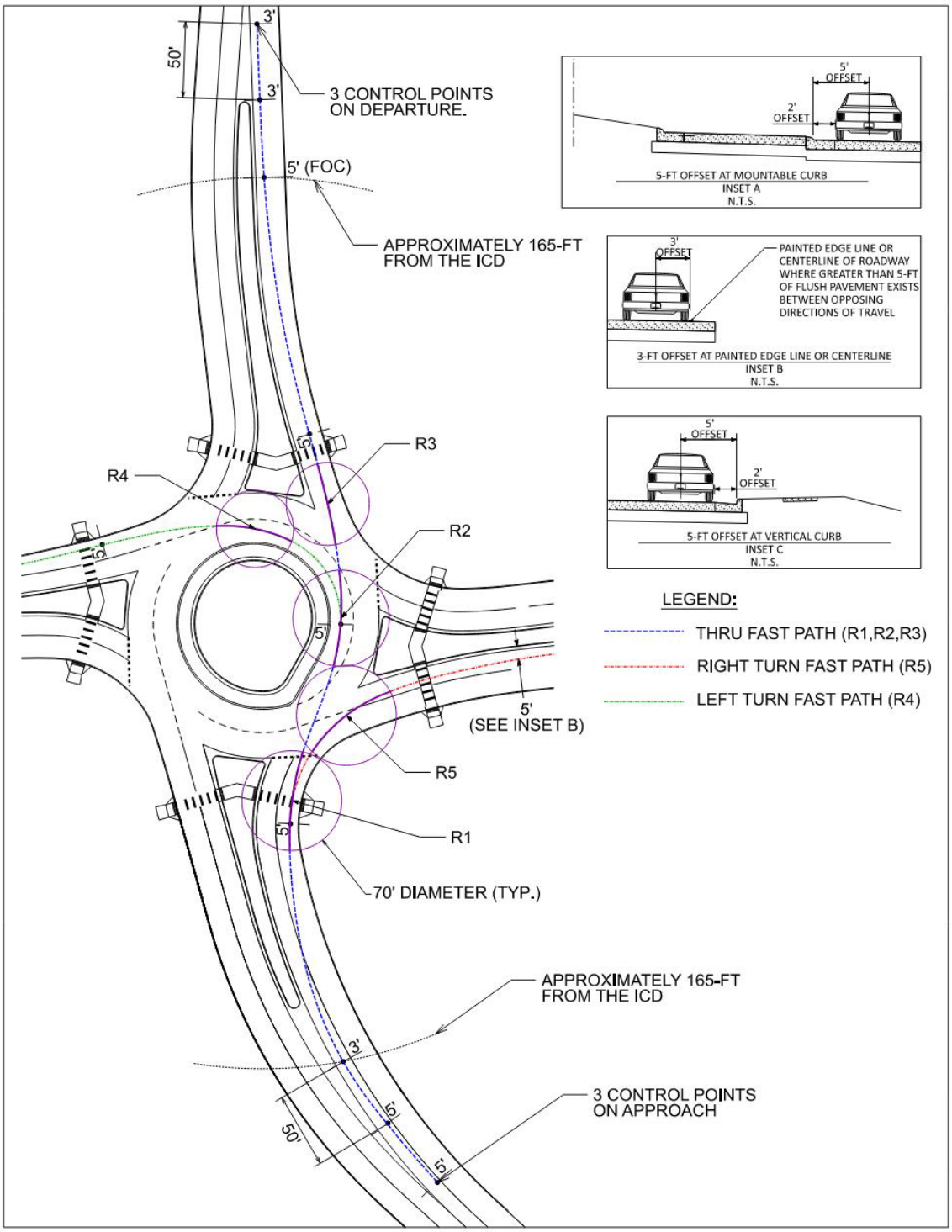


Figure 14-6: Geometric Speed Control Fastest Path Performance Design Checks

14.4.5.3 Path Overlap

Path overlap is a detrimental condition that can arise from inadequate horizontal geometry of multilane roundabout entries and exits, as discussed in **NCHRP Report 1043**, Section 9.6. Providing tangency to adjacent entry and exit lanes has emerged as an important feature of multilane design to discourage vehicle lane changing within the circulating/conflicting conflict zone areas of a roundabout. **Figure 14-7** visualizes the method for checking path overlap. The amount of tangent to avoid entry path overlap and exit path overlap between point A to point B and point B to point C, respectively, is 30-ft to 50-ft and 40-ft to 60-ft. One check of adequate entry geometry is for the tangent point at point A to be located within 5-ft of the entry yield line. Additional tangency beyond 5-ft prior to the entry line should be avoided so the speed-controlling entry radius is not shifted too far in advance of the typical crosswalk location.

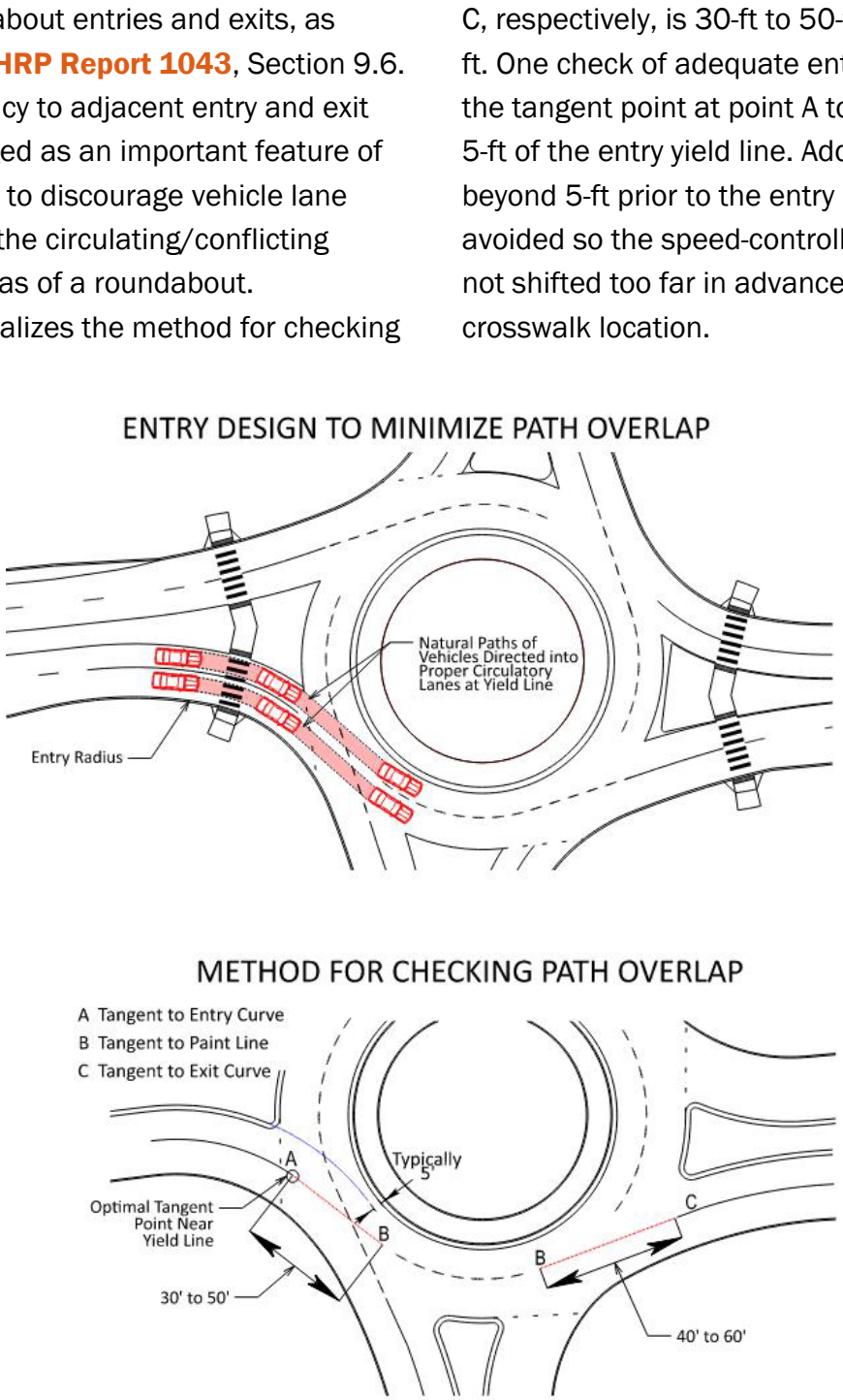


Figure 14-7: Natural Path Performance Design Checks

14.4.5.4 Design and Check Vehicles

NCHRP Report 1043, Section 9.7 covers the topic of design vehicles. The largest anticipated non-articulated vehicle that is needed to be fully accommodated in the circulating roadway within the roundabout (the "design vehicle"), and the largest anticipated articulated vehicle subject to use a given roundabout (the "check vehicle") are key roundabout design parameters. Typical design vehicles are the largest local jurisdiction fire department apparatus or a city bus. An additional check vehicle that includes articulation, such as a WB-40, WB-62TX, or WB-67, should be analyzed in properly sizing entry widths, circulatory roadway widths, truck apron widths, exit widths, and determining the final location of proposed curbing within and adjacent to a given roundabout.

The selection of the design vehicle and the check vehicle to navigate the intersection throughout the life cycle of the intersection is necessary prior to preliminary design. Consideration should be given for individual movements at an intersection. The design vehicle and check vehicle may only need to make thru movements and may not need to make left or right-turn movements on or off the major roadway. Adjacent land use, trip generators, and local and regional routing may factor into the verification of movements of the check vehicle on a case-by-case basis. The selection of the appropriate vehicles should be vetted with TxDOT staff, and possibly with local agency staff, local stakeholders, and potential permitted load routing.

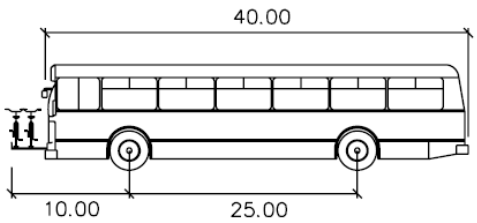
NCHRP Report 1043, Chapter 9 and Appendix A.4 outlines vehicle performance checks for design vehicles and check vehicles.

Key considerations for design vehicles and check vehicles at roundabouts consist of:

- The appropriate design case- straddle lanes or stay-in-lane (refer to **NCHRP Report 10433**, Section 9.7.1 for more information);
- The width of outer lane or inner lane curb offsets;
- If curb and gutter is proposed, the dimensions of curb and gutter are important in keeping design and check vehicle wheel paths off the gutter pan;
- Allowance of a shy distance between the vehicle's outer wheel path and face of curb (or edge of traveled way);
- Analyzing left-turning articulated vehicles within the inner lane of a multilane circulatory to determine the appropriate truck apron width;
- Selection of the largest anticipated vehicle (i.e., the check vehicle) that may need to navigate the roundabout. OSOWs would be classified as a check vehicle if determined to be needed along the route where a proposed roundabout is being considered. For the check vehicle, external truck aprons may be used to facilitate the proper accommodation of an OSOW to avoid widening travel lanes adjacent to and within the roundabout; and
- The use of a WB-62TX check vehicle in lieu of a WB-67 or WB-62 check vehicle.

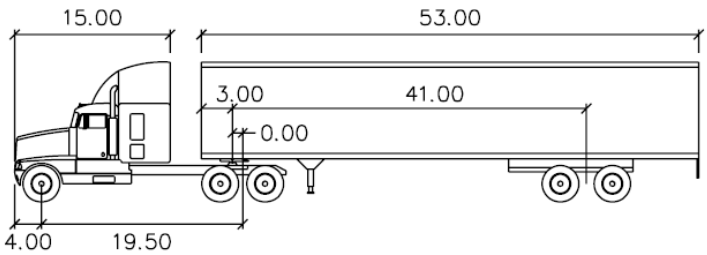
WB-62TX represents a WB-62 with a 53-ft trailer instead of the WB-62's 48-ft trailer. Given that most tractor-trailer combinations will haul 53 ft trailers with their rear tandems moved, it is appropriate to use the WB-62TX at locations other than interstate ramp terminals or near truck terminal locations.

Figure 14-8 shows common design and check vehicle specifications based on **AASHTO's A Policy on Geometric Design of Highways and Streets** for a CITY-BUS and a WB-67. The WB-62TX designation, although not an official designation by AASHTO, is a common tractor-trailer configuration in Texas and is recommended to be used as a check vehicle where a WB-67 configuration is not anticipated. **Section 4.3** of this manual has additional information on Design and Control Vehicles.



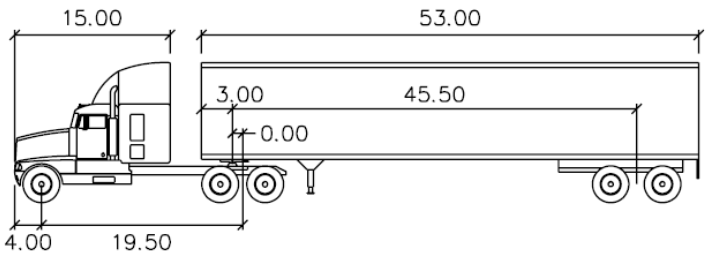
CITY–BUS
feet

Width	: 8.50
Track	: 8.50
Lock to Lock Time	: 6.0
Steering Angle	: 41.4



WB–62TX
feet

Tractor Width	: 8.00	Lock to Lock Time	: 6.0
Trailer Width	: 8.50	Steering Angle	: 28.4
Tractor Track	: 8.00	Articulating Angle	: 75.0
Trailer Track	: 8.50		



WB–67
feet

Tractor Width	: 8.00	Lock to Lock Time	: 6.0
Trailer Width	: 8.50	Steering Angle	: 28.4
Tractor Track	: 8.00	Articulating Angle	: 75.0
Trailer Track	: 8.50		

Figure 14-8: Design Vehicle and Check Vehicle Templates

14.4.5.5 Truck Accommodation at Multilane Roundabouts

Straddle lane design is presumed unless it can be shown that a given intersection is anticipated to accommodate a significant amount of daily or peak period truck traffic. If an intersection is anticipated to have 120 trucks per hour, or more, (one truck every 30 seconds) on a given approach, stay-in-lane design is recommended to be implemented. Contexts for stay-in-lane design will be roundabouts proposed at interchanges, near common OSOW facilities and freight trip generators.

The need for stay-in-lane design increases at roundabouts that are anticipated to experience near- or at-capacity conditions in the peak periods. This is due to the additional time gap needed for large trucks to be able to enter safely into the circulatory while yielding to circulating/conflicting vehicles. A stay-in-lane design in this situation allows passenger vehicles to accept smaller gaps in traffic and proceed through the roundabout while the large truck waits for an extended gap in traffic.

14.4.5.6 View Angle

View angle is the angle created by a driver yielding at the entry line while looking to their left to identify oncoming vehicles that will conflict with their entry path. As the angle that a driver is required to turn their head to identify oncoming traffic increases, vision and detection of potential conflicts becomes increasingly difficult. This may especially be the case with drivers with chronic neck pain, injuries, or elderly drivers. View angle is also a factor with large trucks, whereby the side window placement is commonly a constraint for sight to the left at acute angles.

NCHRP Report 1043, Exhibit A.16 and Figure 14-9 depict the arrangement of vehicles for angle of visibility checks. If an oncoming vehicle's location crosses out of the 105-degree cone of vision for this visibility-to-the-left performance check, drivers will not be able to move close to the yield line for normal sight of oncoming traffic to the left. In such cases, the approach and entry geometry must be modified to improve the view angle. Skewed intersections and right-turn yielding bypass lanes are typical examples where view angle performance checks are vital to providing visibility-to-the-left.

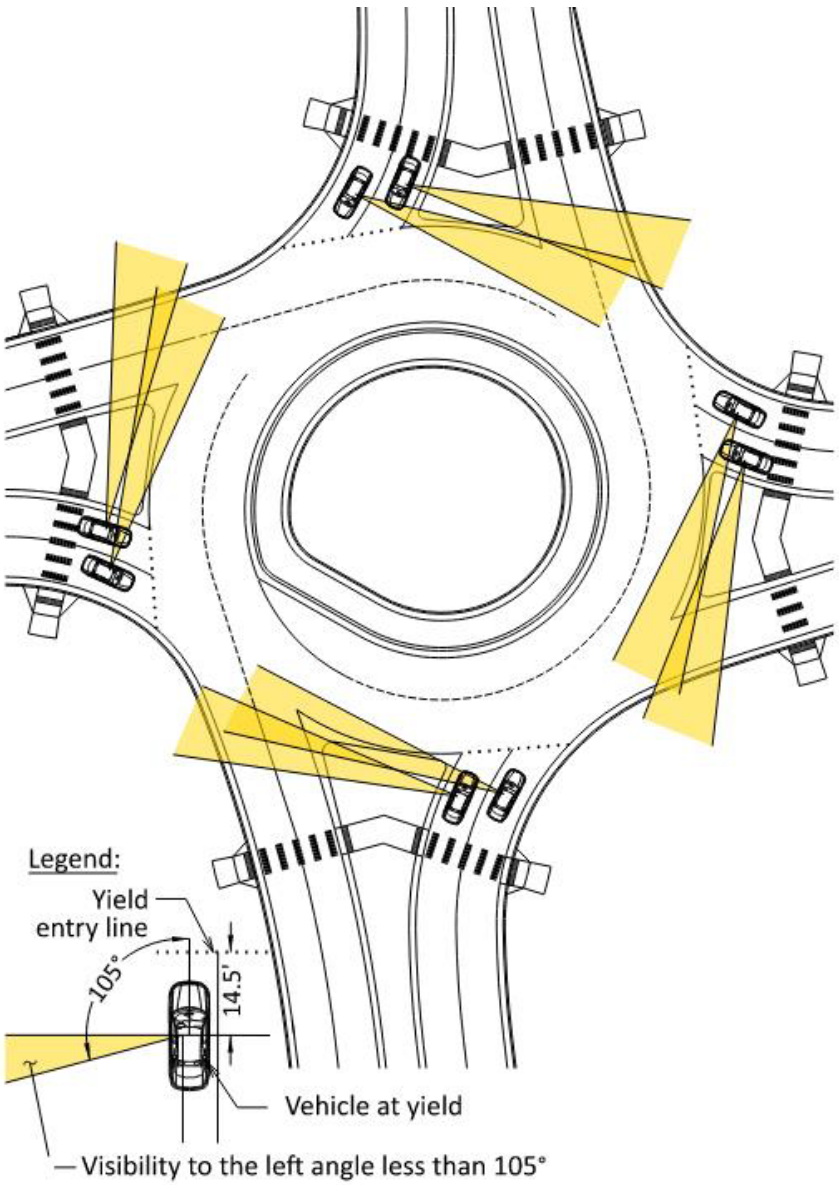


Figure 14-9: View Angle Example

14.4.5.7 Sight Distance and Visibility

NCHRP Report 1043, Section 9.5 documents the relevant sight distance checks for roundabouts. Stopping sight distance, intersection sight distance, and decision sight distance are all relevant to roundabout geometric design. Key considerations for adequate sight distance at roundabouts consist of:

- Proper grading and clear zones within the central landscaped island of the roundabout to accommodate the circulatory roadway's stopping sight distance requirement (refer to **NCHRP Report 1043**, Exhibit 14.9);
- Avoidance of perimeter landscaping, signage, street furniture, franchise utility equipment, and other appurtenances that may restrict sight distance;

- Avoidance of splitter island treatments that may restrict stopping sight distance, especially for approach geometries of skewed intersections and offset left designs. Trees, vertical walls, and franchise utility equipment are a few common obstructions that should be avoided in a splitter island within 50-ft of a roundabout's ICD; and
- Avoidance of an abundance of sight distance. When the central landscaped island of a roundabout is graded flat or lacking berm height, it allows approaching motorists to see through a roundabout. Similarly, if the approaching splitter island and perimeter quadrant to the left of an approaching driver is vacant, it can allow for an approaching motorist to identify the absence of potential circulating/conflicting traffic upon their arrival at the entry of the roundabout. This abundance of sight distance can result in drivers maintaining faster speeds as they recognize no conflicts will be present upon their arrival at a roundabout. Providing restrictive sight elements outside of sight distance envelopes can reinforce speed reduction for approach drivers resulting in safer driving conditions at the crosswalk and entry of a roundabout.

14.4.6 Special Considerations

To supplement **NCHRP Report 1043**, the following roundabout design considerations are presented to increase design flexibility for complicated design context of roundabouts in Texas.

14.4.6.1 Oversized and Overweight Vehicles

Oversized and overweight vehicles are common on Texas roads. Due to the presence of wind turbine tower trailers, heavy construction equipment trailers, and other large transport trucks, it can become necessary to specifically modify a roundabout design to accommodate an OSOW. Coordination should take place between the intersection location's TxDOT Area Office and the engineer-of-record to determine if permitted loads are present at the subject intersection, and if so, the specifications of the largest anticipated vehicle. **NCHRP Report 1043**, Section 10.5.4 discusses the design of roundabouts for OSOWs.

14.4.6.2 High-speed or Intermediate Speed Approach Design

Note for the general purposes of superelevation methodologies, intermediate speeds are classified as between 50-60 mph (see **Table 4-3**). Roundabouts installed in these contexts require recognition of the human factors needs of the driver to detect, recognize, and react to the presence of a roundabout while approaching the intersection at a higher rate of speed.

The primary safety concern in intermediate/high-speed context is clarity of the driving situation, that is, to make drivers aware of the roundabout with ample distance to comfortably decelerate to the appropriate speed. Therefore, designs should abide by these principles:

- Provide the desirable forward stopping sight distance of the entry point based on approach operating speed;
- Align approach roadways and set vertical profiles to make the central island conspicuous and mound the central island 3.5-ft. to 6-ft;
- Splitter islands should extend upstream of the yield line to the point at which entering drivers are expected to begin decelerating. A minimum length of 200-ft is recommended for high-speed approaches;
- It is typical for the approach treatment to extend the project limits to greater than 500-ft from the center of the roundabout. Approach curves should be gentle, become successively smaller and should be sized based on the design speed and expected speed change of 15 to 20 mph per successive horizontal curve; and
- The use of a tangent between horizontal curves is recommended unless the curves have large radii and there is no required superelevation of either curve. A common tangent length is 100ft. The use of a tangent has been shown to eliminate truck rollovers where curve radius and curve length are relatively small in a high-speed context. Refer to the **Innovative Intersections webpage** on TxDOT.gov for additional information.

For approaches to roundabouts see **Table 4-3** for appropriate superelevation methods.

Guidance provided in **NCHRP Report 1043,** Section 10.14 combined with **Figure 14-10** aids in designing splitter island length, horizontal curves, and superelevation to treat roundabouts in an intermediate/high-speed context.

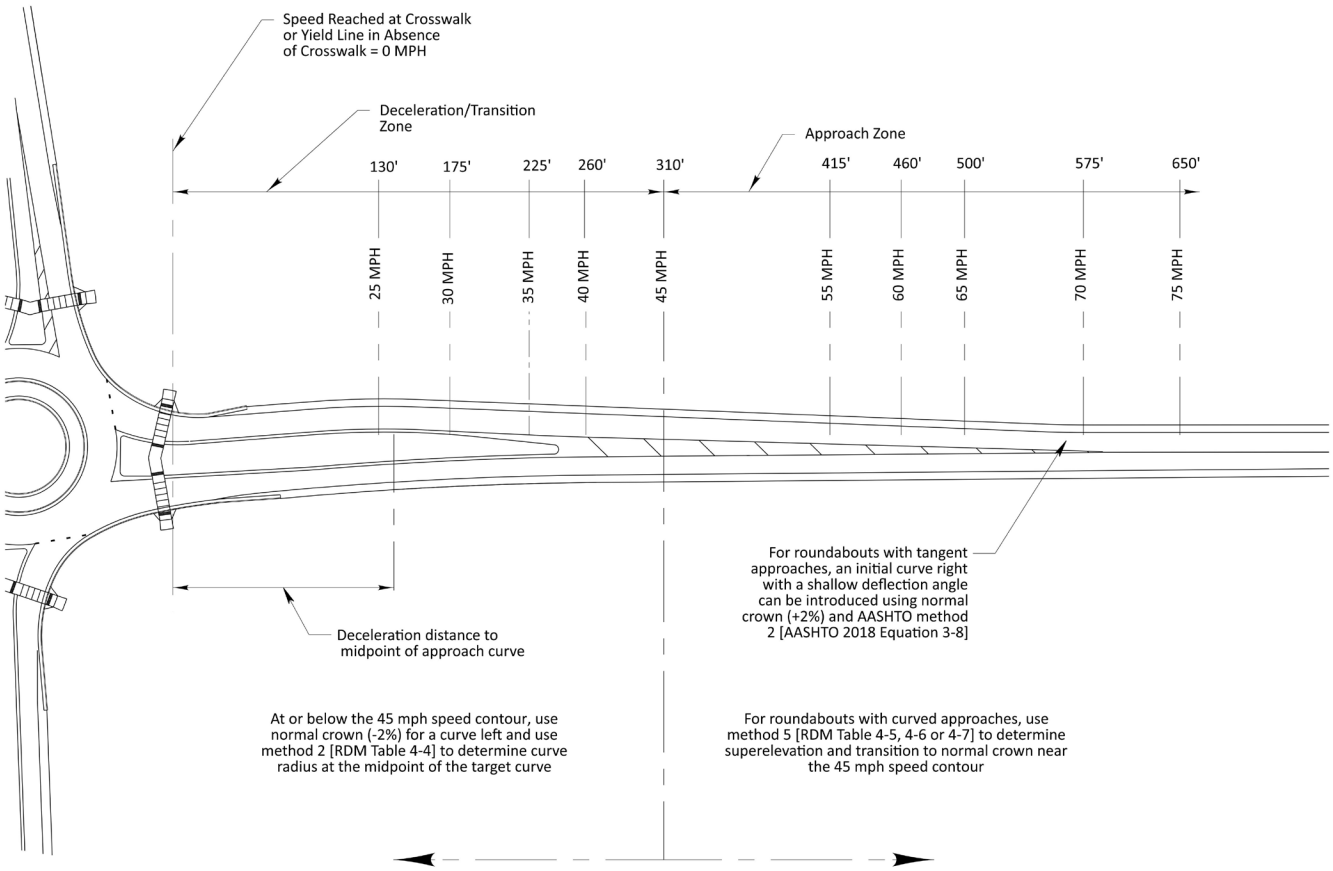


Figure 14-10: High-Speed Approach Design Approach and Deceleration Zones

14.4.6.3 Spiral Design

Spiral circulatory geometry is commonly used when a dedicated left-turn lane is implemented at a multilane roundabout. **Figure 14-11** shows an example of a spiraled movement. Spirals can also be applied for any 2x1 lane configuration roundabout where the single circulating lane spirals to the portion of the roundabout where a resulting two lanes are located within the circulatory. Benefits of using spirals in roundabouts consist of:

- Guiding the circulating vehicle to the outer lane of the two-lane circulatory;
- Where adjacent driveways are located close to the roundabout, or a heavy right-turn movement is common at the adjacent downstream intersection;
- Reduces the size of the conflict zone, the area where entering vehicles interact with circulating/conflicting vehicles; and
- Reinforce yield behavior for the outer lane entering driver that is yielding at

the two-lane entry because the spiral makes it more apparent that the circulating vehicle will cross the entry path of this entering lane.



Figure 14-11: Spiral Design Example

Source: Kyle Kruppa

14.4.6.4 Buffered Lane Design

The concept of buffered lane design has emerged from discussions in the U.S. of how to adapt turbo roundabout design to conventional U.S. multilane roundabout design. Buffered lane design is also emerging as a retrofit solution where constructed roundabouts may have excessive ICDs and/or circulatory widths.

The benefits of buffered lane roundabouts are evident with improved lane discipline for passenger vehicles, reduced vehicle speeds and failure-to-yield conflicts, while also providing lane discipline for articulated trucks. Where existing multilane roundabouts have been built with a surplus of circulatory width, buffered lane design represents a low-cost solution to narrow circulating lanes which in turn enhances speed reduction and lane discipline without having to consider full-depth reconstruction as part of a safety improvement.

Applying buffered lane design to initial installation is done more deliberately while upholding the basic principles of ICD size, geometric speed control, and approach alignment. **Figure 14-12** shows an example of a buffered lane design roundabout.



Figure 14-12: Buffered Lane Example (applied as a retrofit without moving curbs)

14.4.6.5 Turbo Roundabouts

By definition, a turbo roundabout includes a radial entry and exit design, employs orbiting geometry within the circulatory to facilitate the origin-destination path through the roundabout, and makes use of raised dividers to channelize vehicles to enforce in-lane driving. Implementation of turbo roundabouts is in its infancy stage in the U.S. since only a handful of turbo roundabouts have been built in the U.S. Designers must exercise caution when selecting this type of roundabout. Refer to Section 10.7.8 in **NCHRP Report 1043** for additional information regarding turbo roundabouts. An example of a constructed turbo roundabout in the U.S. is shown in **Figure 14-13**.



Figure 14-13: Example Turbo Roundabout (Jacksonville, FL)

14.4.6.6 Expandable Design

Interim and ultimate lane configuration roundabout analysis may result in a preferred method of using future expandable design. If within the operational analysis a roundabout is shown to benefit from a reduced lane configuration on opening day for the first 10 to 15 years of its existence, then expandable design should be considered. Reducing the number of entry lanes on a given approach will result in less conflict points, shorter crossing distances for pedestrians and bicyclists, and provide for safer operations.

NCHRP Report 1043, Section 10.8 discusses the use of expandable design, including

commentary on the advantages and disadvantages of expanding outward versus expanding inward.

14.4.6.7 Railroads and Preemption

NCHRP Report 1043, Section 12.7 provides guidance on roundabouts near rail crossings. Roundabouts with at-grade rail crossings nearby or within the roundabout have been constructed at many locations in the U.S., including downtown Daingerfield, TX. The decision of whether to install crossing gates at a roundabout may be considered early in the planning or design process. Early and up-front coordination is highly recommended with the governing railroad agency if a roundabout is proposed near an at-grade rail crossing. Risk analysis, traffic queue calculations, train crossing frequency and duration, and proximity of the roundabout to the rail crossing are important factors when considering a potential roundabout installation in this type of context.

14.4.6.8 Median and Access Management Benefits and Considerations

Various topics arise when considering the effect of roundabouts and access management. Roundabout corridors can replace median-opening cross-access along an undivided or divided roadway. Motorists can make use of U-turn movements at a roundabout to facilitate what would previously be a left-turn movement to access a property on the opposite side of the street.

A corridor of roundabouts combined with continuous raised medians provides amplified safety benefits by eliminating high severity crash types such as head-on and right-angle or side-impact collisions. Roundabouts provide U-turns for all vehicle types at low speeds, eliminating the risks of mid-block left-turns and U-turns across opposing high-speed traffic. Median width can thereby often be reduced since mid-block turning movements are prohibited. This strategy is called “wide nodes and narrow roads”. Severe crash risks are reduced within the narrowed mid-block roadway section and at the intersections. Roundabout spacing in this type of corridor is important to consider to limit the distance required for a vehicle to make a U-turn to access adjacent property.

Driveway locations relative to a roundabout can typically be moved closer to the intersection with a roundabout, as compared to a conventional traffic signal installation, due to shorter queue lengths. Other factors influencing the location of a driveway relative to a roundabout consist of:

- The location of a bike ramp or crosswalk that may conflict with the driveway;
- A queue spillback analysis at the entry of a roundabout;
- The frequency of left-turns wanting access to the driveway; and
- The resulting storage length for a yielding right-turn bypass lane.

14.4.7 Preliminary and Final Design

14.4.7.1 Pavement Design

Determination of a concrete or asphalt pavement section is important to the schematic and preliminary design of a roundabout due to the subsequent

performance checks for fastest path and design vehicles. Determining whether the roadway will have a monolithic curb, or a separate curb and gutter section may have a direct impact on the curb offset location and the subsequent performance checks performed for a roundabout.

14.4.7.2 Vertical and Grading Design Best Practices

For constructability and grade control, setting up additional alignments and profiles of the edge of pavement produces optimal grade control for retrofit designs. Developing a grading surface that closely follows existing topography or the natural grade of the intersection often results in a tilted plane, also known as "tipping the circle". Tipping the circle requires gradual transition from sloping in on the high side to sloping out on the low side. The preferred maximum cross-slope of the low side should be 2-percent to prevent truck roll-overs.

Additional considerations for the proper accommodation of OSOWs may be needed for vertical clearance checks of lowboys and other vehicles that have a reduced vertical envelope. In rare cases, the height of the center island truck apron and the circulatory grade may need to factor into the check vehicle vertical clearance.

For additional guidance, refer to **NCHRP Report 1043**, Chapter 11.

14.4.7.3 Drainage

Ideally, drainage structures will be installed outside of the limits of the ICD of a roundabout. Inlets upstream of crosswalks in advance of a roundabout and downstream of the ICD typically provide enough capture from runoff within the footprint of a roundabout. Minimum and maximum slopes should be analyzed, including a typical cross slope within the circulatory roadway of 1.5 percent to 3.0 percent. Chapter 10 of **NCHRP Report 1043** provides guidance on drainage best practices and examples.

14.4.7.4 Pavement Markings

Pavement marking plans should be separated from signing plans for clarity. Legends are encouraged on pavement marking sheets to convey the TxDOT bid item numbers and an accurate description of the bid item, including color, width, length, and gap length of the pavement marking.

Figure 14-2 to **Figure 14-5** represent TxDOT's preferred pavement markings for roundabouts. **NCHRP Report 1043**, Chapter 12 contains supplemental information regarding pavement markings at roundabouts. The **TMUTCD** also serves as a resource in preparing pavement marking designs. Key considerations for pavement markings at roundabouts consist of:

- The use of standard arrows in lieu of fish-hook arrows (**NCHRP Report 1043,** Exhibit 12.6). TxDOT's preference is to use traditional arrows for all approach and circulating area lane use arrows;
- Worded or route destination shield symbols used at interchanges or other designated facilities;
- The selection of circulating lane separation line types for multilane

roundabouts. Several different methods of marking the circulatory roadway have emerged in roundabout guidelines and in practice, including a solid-dotted combination (**NCHRP Report 1043**, Exhibit 12.26), a dashed consistent pattern (**NCHRP Report 1043**, Exhibit 12.27), and a double solid lane pattern (a.k.a "buffered lane design). (**NCHRP Report 1043**, Exhibit 12.28).TxDOT's preference is to make use of the dashed consistent pattern as shown in **Figure 14-4**;

- The use of yield triangles (a.k.a. yield line) in lieu of YIELD words at the entry points of each lane. TxDOT's preference is to use yield triangles in lieu of YIELD words. The yield triangles shall be placed where the top of the triangles are perpendicular to the travel lane;
- The widths of entry line markings and circulatory pavement markings. Pavement markings, at minimum, must be 6 inches adjacent to and within a roundabout. Some marking types will have increased widths of 8 inches (lane edge lines) up to 36 inches (entry yield line); and
- The use of contrast markings on newly constructed concrete pavement to improve the visibility of the markings is required.

14.4.7.5 Signage

The most current version of the **TMUTCD** must be sourced for providing signage at roundabouts. Key considerations for signage at roundabouts include:

- Mount the R6-4 series regulatory sign within the central island at a height equal to the bottom of sign being 48-in from the finished elevation of the circulatory pavement adjacent to the truck apron curb;
- D1-5 Exit Destination signs should be considered for on-system non-urban roadways;
- If a splitter island yield sign is installed, angle the face of the sign perpendicular to a vehicle a distance equal to the SSD on the approach to the roundabout; and
- Angle the face of the yield sign that is located behind the perimeter curb perpendicular to the adjacent curb.

14.4.7.6 Overhead Signage

Advance overhead lane assignment signage on multilane roundabouts is strongly recommended for wayfinding and driver recognition, especially at interchange roundabouts and roundabouts that are located near other alternative intersections, interstate access points, or where more than two lanes may exist along the approach to a roundabout. Exhibit 12.8 of **NCHRP Report 1043** provides an example of the preferred lane assignment sign assembly and placement for a two-lane roundabout.

14.4.7.7 Illumination

NCHRP Report 1043, Sections 14.1, 14.2 and 14.3 focus on illumination considerations, lighting levels, and illumination equipment type and location. Since agency illumination policies vary widely across the State, it is important to

factor in local restrictions or requirements such as Dark Sky Initiatives.

14.4.7.8 Landscaping

Section 14.4 of **NCHRP Report 1043** provides guidance for roundabout landscaping. Key considerations for landscaping include:

- Avoid vertical obstructions being placed within the sight distance envelopes calculated based on **NCHRP Report 1043**, Section 9.5. This includes walls and rigid objects within the central landscaped island, the splitter island (within 50-ft of the entry point), and on the perimeters of the roundabout where front vehicle overhang and rear tandem overtracking may require the use of this space;
- Follow the berm guidance shown on Exhibit 14.9 of **NCHRP Report 1043**. A raised central landscaped island with finished ground elevation 3.5-ft to 6.0-ft above the circulatory roadway elevation is highly recommended; and
- Avoid excessive sight distance and introduce visual blockage outside of calculated sight distance envelopes. Excessive sight distance may encourage drivers to avoid deceleration approaching a roundabout as a driver may identify the absence of circulating/conflicting traffic hundreds of feet in advance of the entry point. The resulting high speed may result in a single-vehicle crash due to driving above the intended speed of the roundabout, or a higher-severity crash if a conflict goes unseen up to the conflict point.

14.4.7.9 Construction Phasing

For roundabout installations where an existing intersection is being reconstructed or modified to accommodate a roundabout, attention to a few factors that influence how the roundabout will be constructed is important. Common considerations consist of:

- Buffers from existing permanent structures or equipment to temporary travel lanes;
- Space for stockpiling and staging of materials and equipment;
- Vertical elevation changes within the limits of construction. This tends to promote keeping the proposed pavement elevations close to existing;
- Traffic routing strategies through the work zone. If the work zone will be closed to traffic with full detours, partial traffic, or full traffic operations;
- Buffer zones between the limits of each phase of construction and adjacent traffic;
- The preparation of a jointing plan that includes the corresponding location of construction joints; and
- The temporary routing and treatment of storm water runoff within construction phases.
- Determining the adequate widths needed for large trucks during traffic handling operations of each phase.

Oftentimes opposing directions will be narrowly separated by a double-yellow pavement marking. If routing bi-directional traffic through a portion of a constructed multilane roundabout, additional width may be needed to accommodate the swept path of an articulated truck or front/rear overhang of an extended vehicle.

14.4.7.10 Concrete Pavement Jointing

For concrete pavement roundabouts, a jointing plan is commonly prepared by the engineer-of-record or supplied by the contractor. Multilane roundabouts should have panels and joint lines such that a pinwheel pattern is developed to favor yielding of entering traffic and priority to existing traffic. Joint and panel patterns must not contradict lane divisions and pavement markings. The optimal jointing plan reveals where lane divisions are and where vehicles circulating can exit a roundabout without changing lanes.

Exhibit 13.8 of **NCHRP Report 1043** and **Figure 14-14** show the use of a pinwheel pattern jointing plan. Joints must also be installed using conventional minimum and maximum panel sizes. Utility appurtenances, such as manholes, water valves, and curb inlets, should be accounted for in jointing techniques, to minimize cracking and failure of adjacent concrete pavement.

For roundabouts built under traffic conditions, the jointing plan must factor in the location of construction joints based on the phased construction of the roadway. Construction phasing considerations may result in a deviation from a standard jointing pattern, however, the proposed jointing strategy should place longitudinal and transverse joints as close as possible to a jointing design that would be produced in lieu of phased construction.

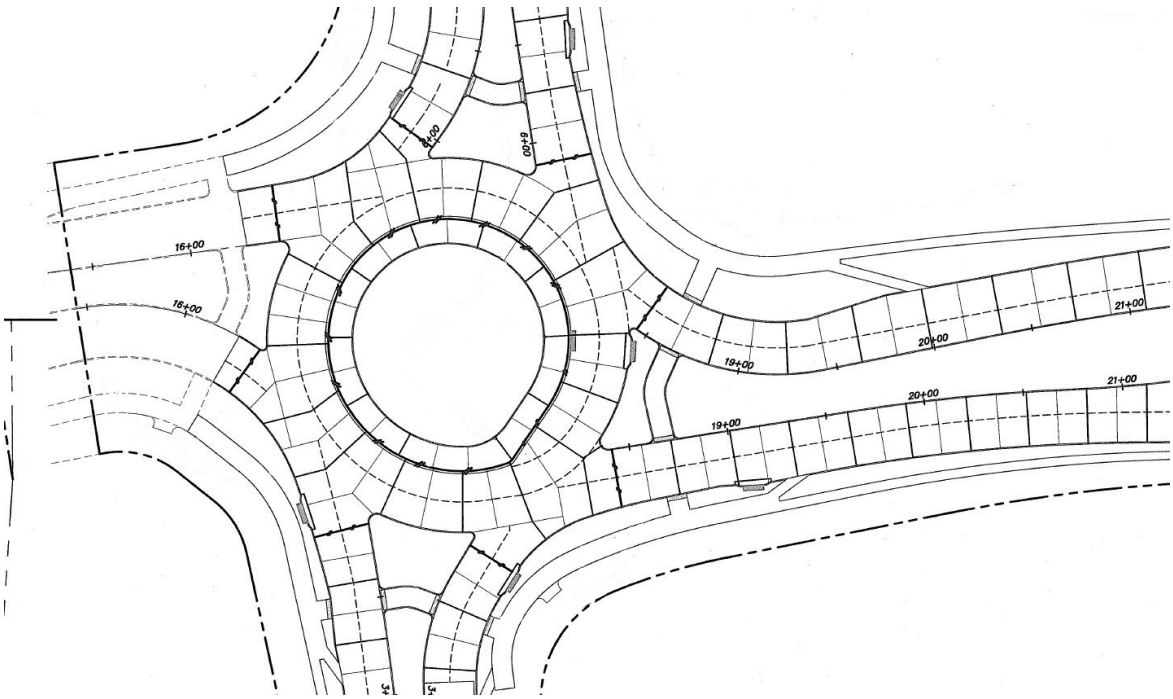


Figure 14-14: Example Pinwheel Jointing Pattern at a Multilane Roundabout

14.4.7.11 Bike Ramps

Bicycle ramps can be beneficial at single-lane roundabouts and are commonly considered at multilane roundabouts where on-street bicycle activity exists or is anticipated. **NCHRP Report 1043**, Exhibit 10.25 and Exhibit 10.26 show an example of how bikes can transition from the on-street facility to the parkway facility and from the parkway facility to the on-street facility.

14.4.7.12 Pedestrian Crossing Devices

Based on the **August 2023 Federal Register Vol. 88, No. 151 Rules and Regulations** publication, multilane pedestrian crossings at roundabouts and channelized turn lanes must have additional treatments that alert motorists to the presence of pedestrians or slow or stop traffic at those crosswalks. Additional treatments can consist of raised crosswalks, pedestrian actuated rectangular rapid flashing beacons (RRFB), pedestrian hybrid beacons (PHB), or a traffic control signal with a pedestrian signal head (R306.4.2).

The August 2023 Rules and Regulations update included additional clarification and guidance for pedestrian facilities consisting of:

- Detectable warning surfaces should be provided at all driveways where the driveways are controlled with stop or yield control (R205.7);
- The cross slope of the pedestrian access route within a crosswalk at a roundabout is permitted to be the same as the grade of the street that it crosses, up to a maximum slope of 5.0% (R302.5.2.2, R302.5.2.3, R302.5.2.4); and

- Where pedestrian crossing is not intended, the pedestrian circulation path shall be separated from the curb, crosswalk to crosswalk, with landscaping or other nonprepared surface 24 inches wide minimum (R306.4.1.1).

14.5 Continuous Green T-Intersection (CGT)

14.5.1 Overview

A Continuous Green-T (CGT) is a T-intersection design where one major street direction of travel (the top side of the "T") can pass through the intersection without stopping and opposite major street direction of travel is typically controlled by a traffic signal.

Left-turn vehicles from the side street use a channelized receiving lane on the major street to merge onto the major street. A CGT intersection is typically signalized but can also be designed without a traffic signal.

Intersections of this type are common and have performed well in other states including Florida, Utah, Nevada, and New York. CGT intersections reduce delay at high-volume locations by removing the need to stop both directions of mainline traffic to accommodate left-turns.

A CGT is also known as:

- Turbo-T intersection;
- High-T intersection; and
- Seagull intersection.

Figure 14-15 shows an example of a GCT intersection.

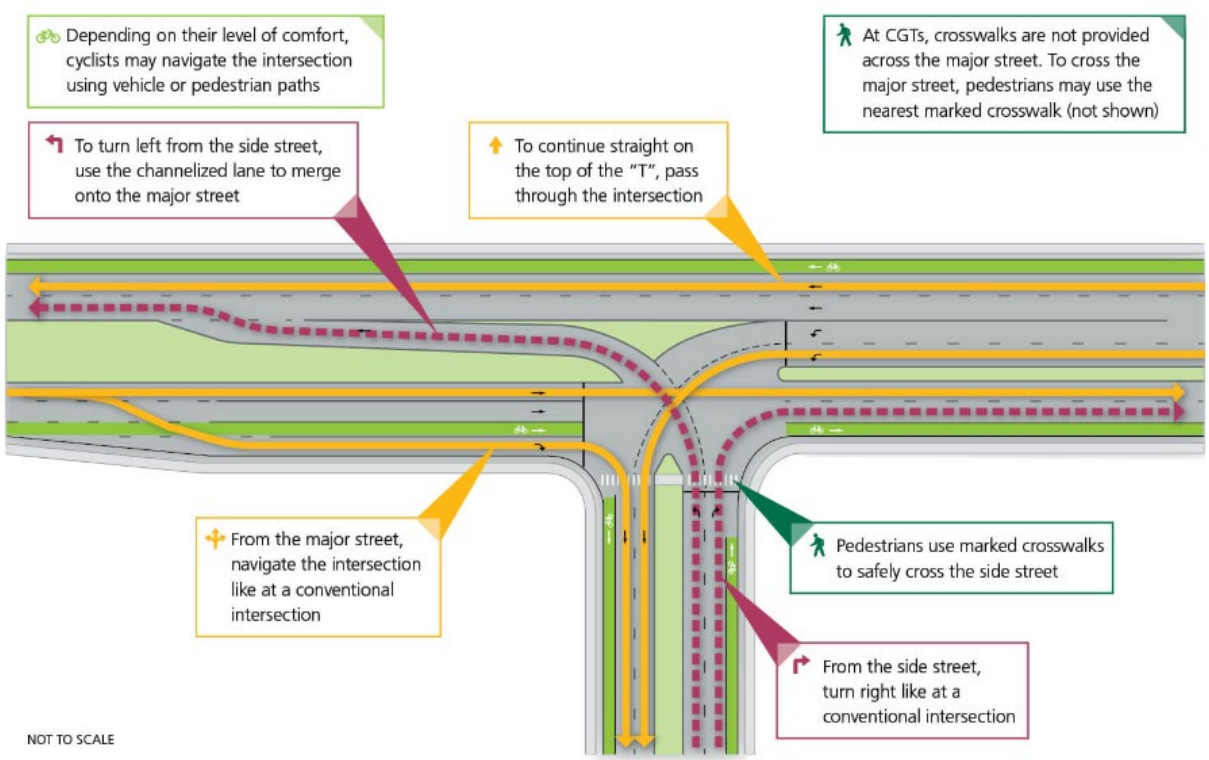


Figure 14-15: Example of a CGT Intersection
Source: Innovative Intersections – Continuous Green-T (CGT) (VDOT)

Benefits of a CGT include:

- **Improved safety** – channelization of left-turn vehicles from the side street reduces the potential for angle crashes.
- **Increased efficiency** – because one direction of travel on the major street is free-flow, more green time can be provided to the other movements, reducing delay at the intersection.
- **Free-flow in one direction** – one direction of travel on the major street never stops, which improves traffic signal synchronization and reduces corridor travel times.

Conditions where a CGT may not be suitable include:

- If there is another intersection beyond the CGT that experiences congestion. This may cause spillback into the CGT and prevent the resultant free-flow;
- Intersections where pedestrians need to cross the major street; and
- Highly developed areas. The CGT may restrict access to existing driveways on the continuous flow lanes.

14.5.2 Design Considerations

A CGT should be considered in the following instances:

- Intersections with three legs;
- Heavy through traffic volumes on the major street;
- Moderate to low left-turn traffic volumes on the side street;
- No driveways along the major street opposite the side street; and
- A limited number of pedestrian crossings across the major street or with an alternative pedestrian crossing location nearby.

Refer to **FHWA Safety Evaluation of Continuous Green T Intersections** for more information on the safety performance of the CGT intersection.

14.5.3 Sight Distance

Sight distance for a CGT is similar to a standard intersection. 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.

14.5.4 Access Management

From an access management standpoint, the CGT intersection will restrict access to driveways near the intersection for vehicles turning left into the top side of the T. Driveways with parallel roads may be provided beyond the intersection to provide access. Driveways near the intersection may also cause weaving.

14.5.5 Pedestrian and Bicyclist Considerations

Due to the continuous flow of traffic on the top side of the T, a CGT does not typically allow pedestrians to cross the major road. Traffic signal equipment can be installed to stop traffic

along the continuous green cycle to accommodate pedestrians and bicyclists. If necessary, provide crosswalks and adequate sight distance and lighting for safety. Bicyclists may navigate the CGT either at the crosswalks provided, or by using bike lanes along the major road.

14.6 Restricted Crossing U-Turn Intersection (RCUT)

14.6.1 Overview

The RCUT, also known as a J-turn or superstreet intersection, is an at-grade intersection with directional medians such that all minor road traffic makes a right-turn followed by a U-turn back to cross a designated location which can be either signalized or unsignalized. The benefits of the RCUT include a significant reduction in intersection conflict points, and a reduction in crash rates and severity. TxDOT has adopted the **FHWA Restricted Crossing U-Turn Intersection Informational Guide** as the primary source for RCUT Guidelines. The information contained in this section is considered a companion guide and is intended to document TxDOT's suggested approach to RCUT design.

See **Figure 14-16** through **Figure 14-18** for examples of RCUTs with signals, without signals, and with merges. For RCUT 3-leg approaches, refer to Chapter 7 of the **FHWA Restricted Crossing U-Turn Intersection Informational Guide**.

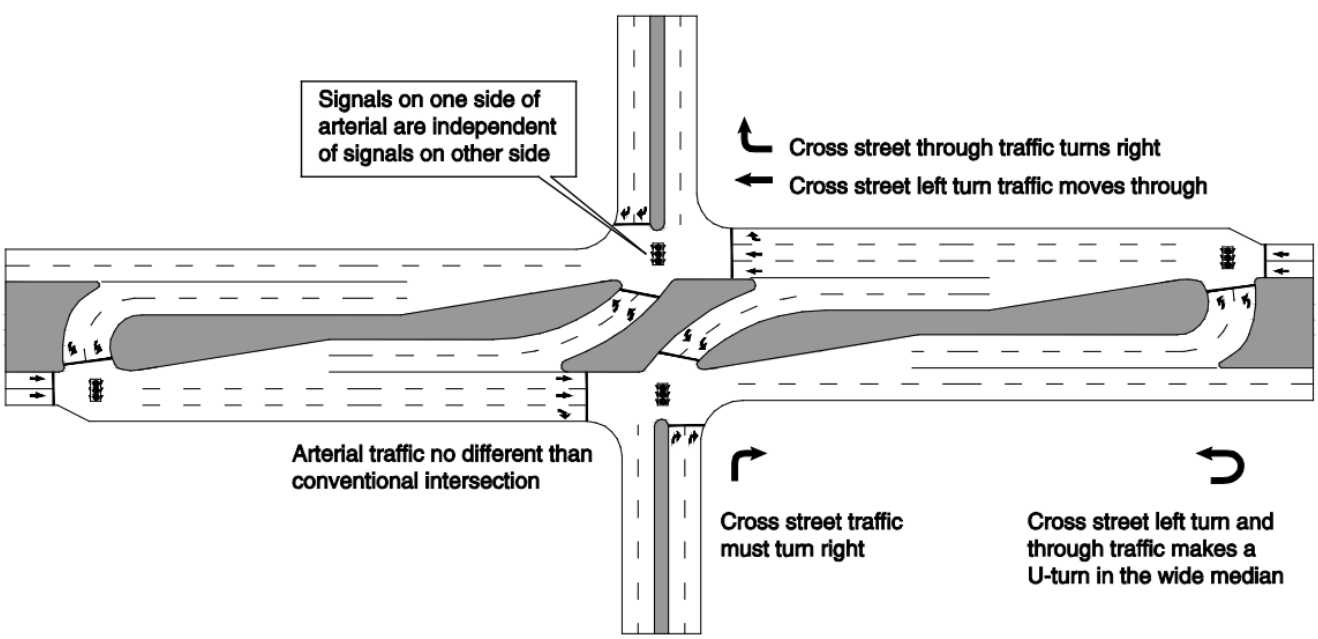


Figure 14-16: Example of RCUT Intersection with Signals
Source: FHWA RCUT Informational Guide

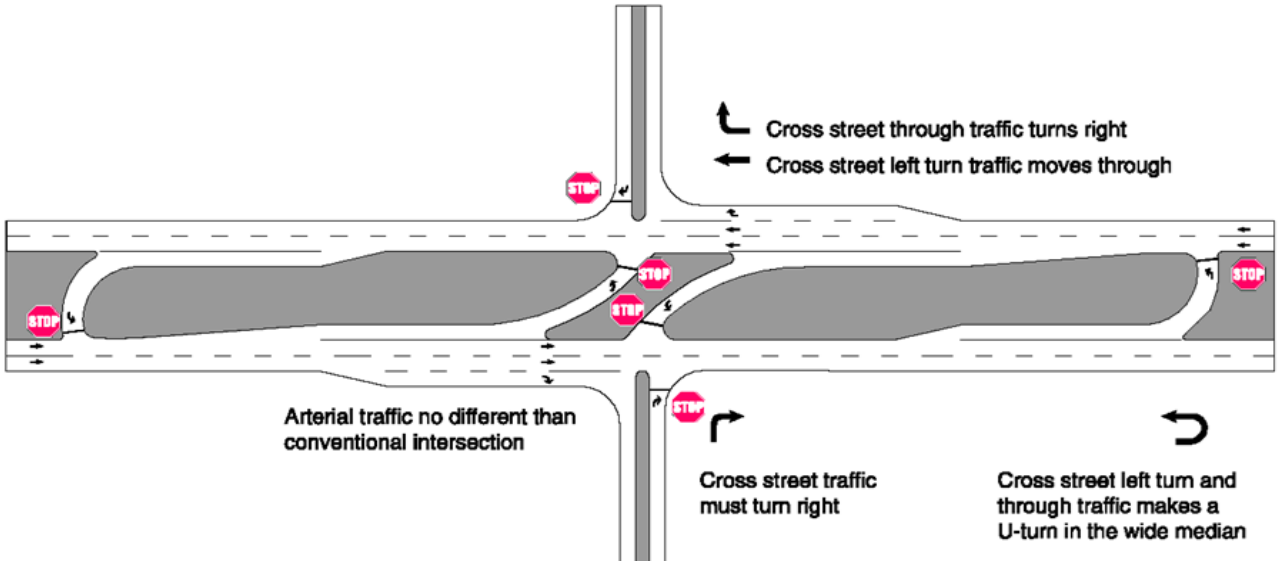


Figure 14-17: Example of RCUT Intersection with Stop-Control
Source: FHWA RCUT Informational Guide

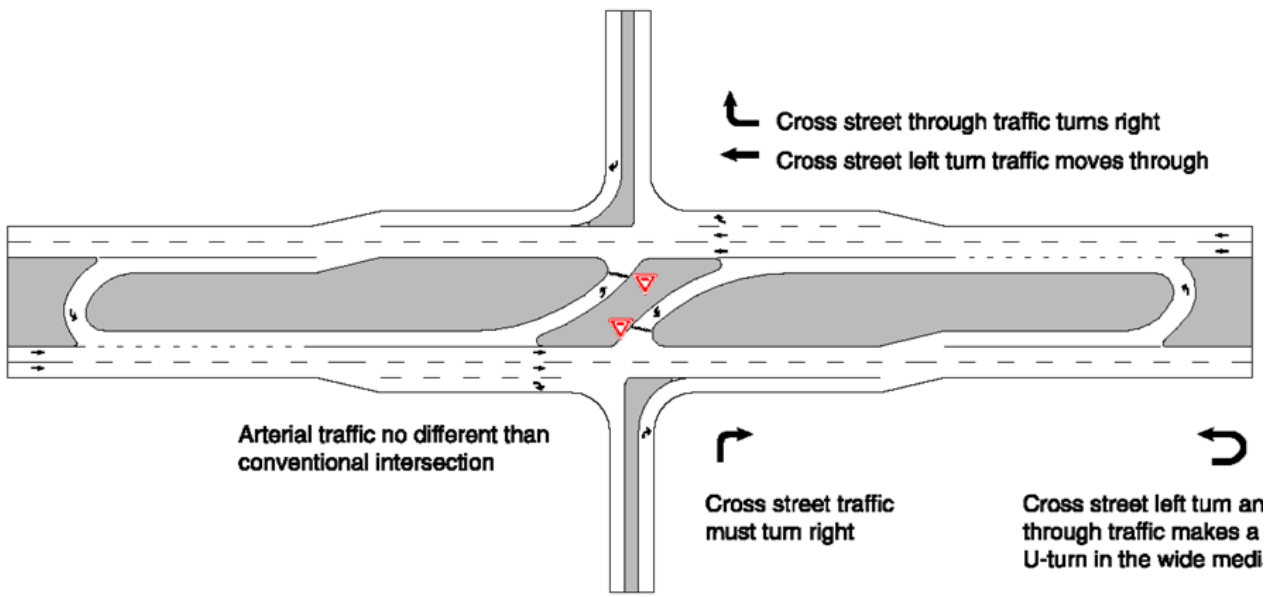


Figure 14-18: Example of RCUT Intersection with Merges
Source: FHWA RCUT Informational Guide

Applications of an RCUT intersection include minor road intersections along rural four lane divided highways, as an alternative to signalization at select intersections to maintain throughput of the main highway, as a corridor treatment along signalized routes to minimize travel times, and as an interim alternative to constructing a grade separated interchange. RCUTs may not be suitable at locations where the minor crossing street has a high volume of left-turn movements, or where there are multiple private drives near the intersection.

14.6.2 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-ft to 800-ft. downstream of the minor road.

The median width is a function of the design vehicle (typically a WB-62TX or WB-67) and the preferred alignment of the design vehicle as it turns into the opposing traffic; see **Chapter 13** for 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 14-19** and **Figure 14-20** 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.

Figure 14-20 illustrates an RCUT scheme showing the extension of left-turn lane deceleration for the left-turn lane and for the acceleration lane after a vehicle exits the U-turn loon. These features are preferred for slow moving vehicles such as trucks and farm equipment that cannot accelerate and decelerate in the distance available between the RCUT and the U-turn. This design concept also affords placement of the U-turn closer to the RCUT. Making it possible to compress the overall footprint with potential space and cost savings.

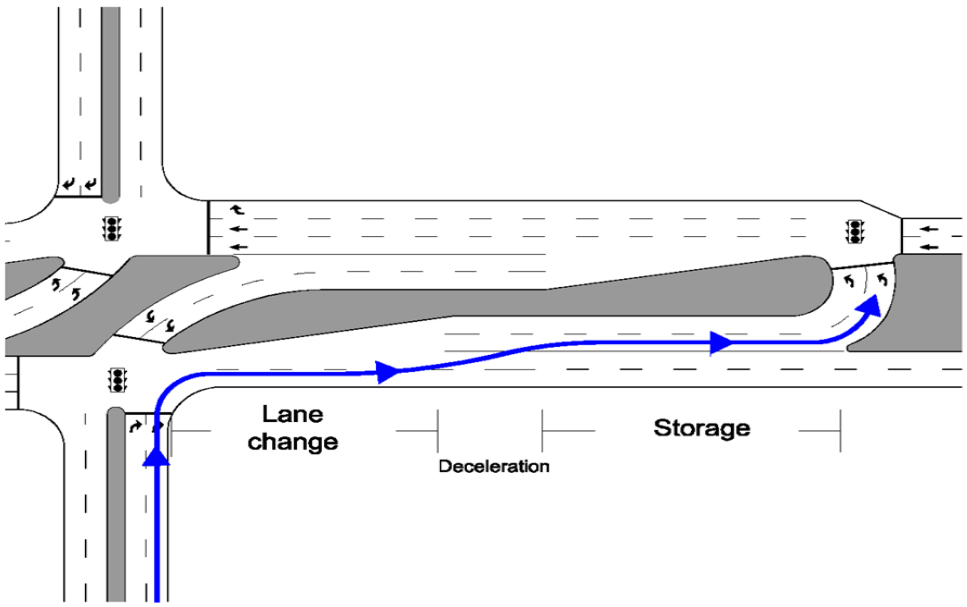


Figure 14-19: Spacing Consideration for a Minor Street Through or Left Movement
Source: FHWA RCUT Informational Guide

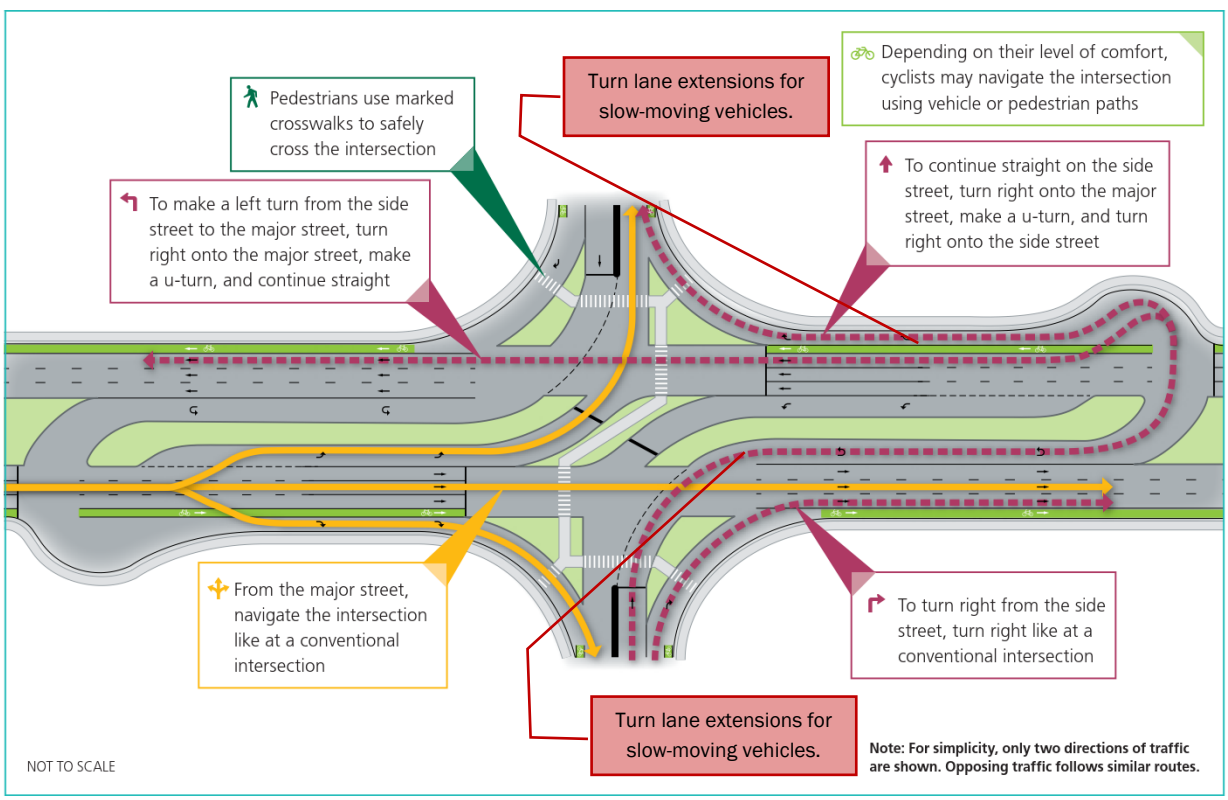


Figure 14-20: Spacing Consideration for a Minor Street Through or Left Movement

Source: FHWA RCUT Informational Guide

Source: VDOT website, accessed March 5, 2024 (used by permission)

14.6.3 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 14-21** demonstrates this concept.

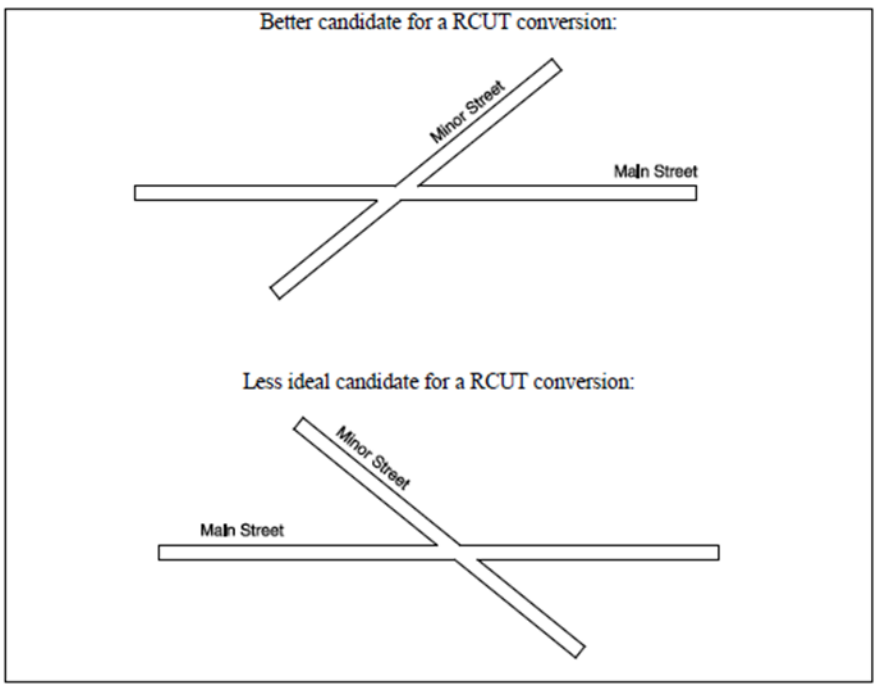


Figure 14-21: RCUT Intersection Angle Considerations

14.6.4 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) by simplifying vehicular movements and eliminating left-turns. However, pedestrians and bicyclists traversing an RCUT intersection have safety challenges as well, similar to a MUT intersection. 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 14-22**. The **FHWA Restricted Crossing U-Turn Intersection Informational Guide** provides guidance for additional pedestrian crossing alternatives.

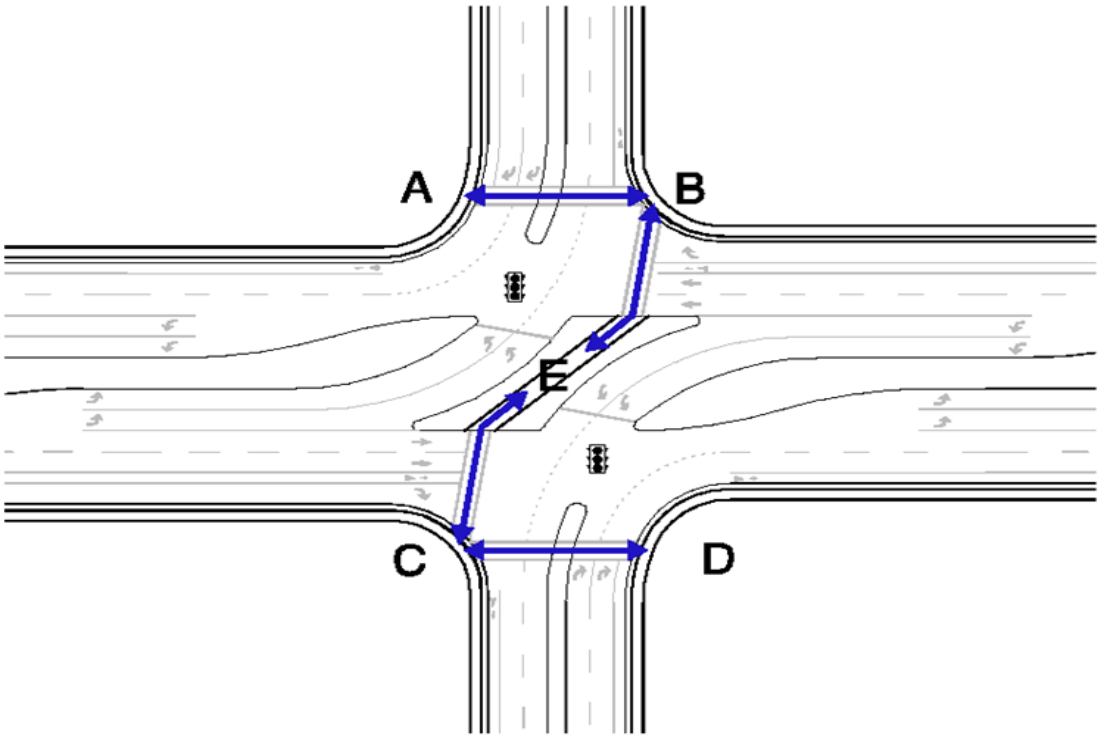


Figure 14-22: Pedestrian Movements in a RCUT Intersection
Source: FHWA RCUT Informational Guide

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; and
- Bicyclists following vehicle rules (legal but undesirable option): 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 14-23** depicts the three options.

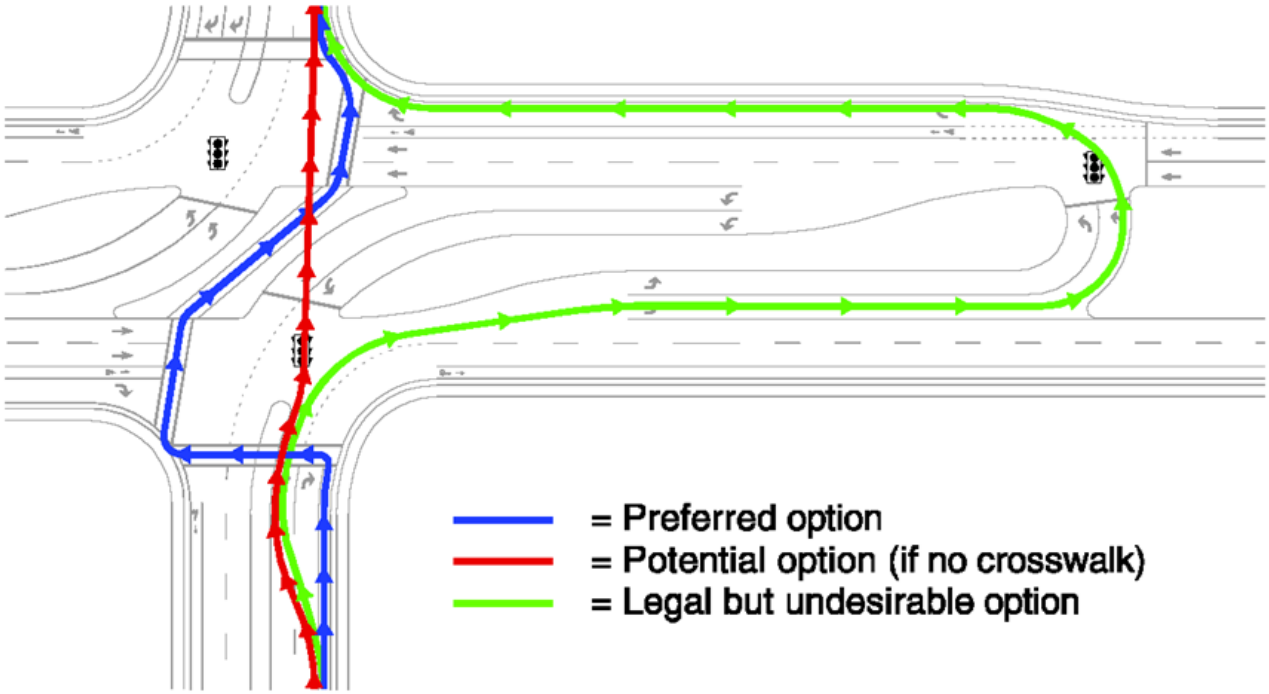


Figure 14-23: Minor Street Through Options for Bicycles
Source: FHWA RCUT Informational Guide

14.6.5 Access Management

From an access management standpoint, possible advantages of an RCUT include the following:

- Can provide a U-turn movement for multiple side streets along the RCUT corridor;
- Allows flexibility for crossover locations to accommodate adjacent driveways and side streets; and
- Provides significant progression benefits along the corridor, which can allow for speed control using the signals.

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. Refer to the **FHWA Alternative Intersections/Interchanges Informational Report (AIIR)** for additional guidance with respect to access management.

14.6.6 Safety

RCUT intersections have safety benefits due to reduced conflict points, especially crossing conflict points, resulting in fewer severe crashes. Potential safety issues to consider at RCUT intersections include:

- Conflicts where U-turns are aligned with driveways;
- The need for channelization to prevent wrong-way movements;
- Where U-turns are made with high-speed opposing vehicles;
- The increased chance of deceleration lane spillback; and
- The potential for weaving movements on the major street.

For additional information on safety refer to the **FHWA Restricted Crossing U-Turn Intersection Informational Guide**.

14.6.7 Operations

RCUT intersections may be signalized or unsignalized. It is only recommended to implement an unsignalized RCUT at a rural intersection. One unique operational benefit of a signalized RCUT is that the signalization of each direction of travel on the major road can operate independently. For more information on operations refer to the **FHWA Restricted Crossing U-Turn Intersection Informational Guide**.

14.6.8 Traffic Control Devices

RCUT intersections require clear traffic control devices to guide drivers through the intersection. The four main objectives of traffic control devices in RCUT intersections are:

- Provide signage and pavement markings to prohibit left-turns and alternative routing of left-turn movements;
- Provide one-way and wrong-way signage;
- Provide a means for safe bicycle and pedestrian crossings; and
- Provide appropriate safety lighting at conflict points.

Refer to the **FHWA Restricted Crossing U-Turn Intersection Information Guide** for further guidance on traffic control devices. The **TMUTCD** remains the primary source for traffic control device standards.

14.7 Displaced Left-Turn Intersection (DLT)

14.7.1 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** as the primary source for DLT Guidelines along with TxDOT's **Continuous Flow Intersection Fact Sheet**.

The information contained in this section is considered a companion guide and is intended to document TxDOT's suggested approach to DLT design. See **Figure 14-24** for a typical four-legged DLT with displaced lefts on a major street.

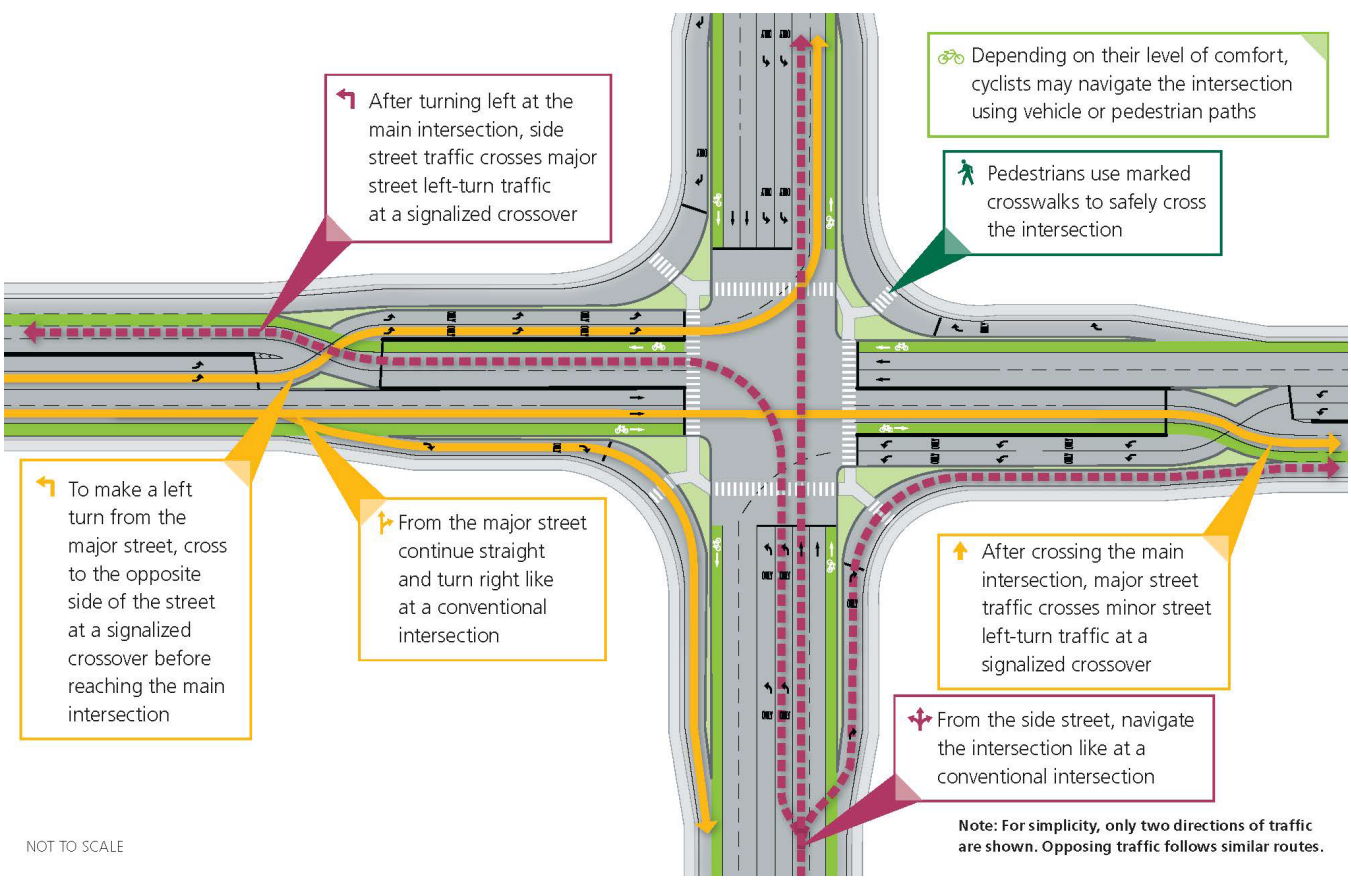


Figure 14-24: Four-legged DLT with Displaced Lefts on a Major Street
Source: VDOT

Benefits of a DLT include a reduction in the number of signal phases with a resultant increase in vehicle throughput. See **Figure 14-25** for an example of a DLT Intersection with displaced left-turns on all approaches. Conditions where a DLT may not be suitable are:

- Areas with limited ROW, due to the large footprint required for a DLT;
- Where there is a lack of driver familiarity and no driver education is provided; and
- Intersections with nearby adjacent driveways.

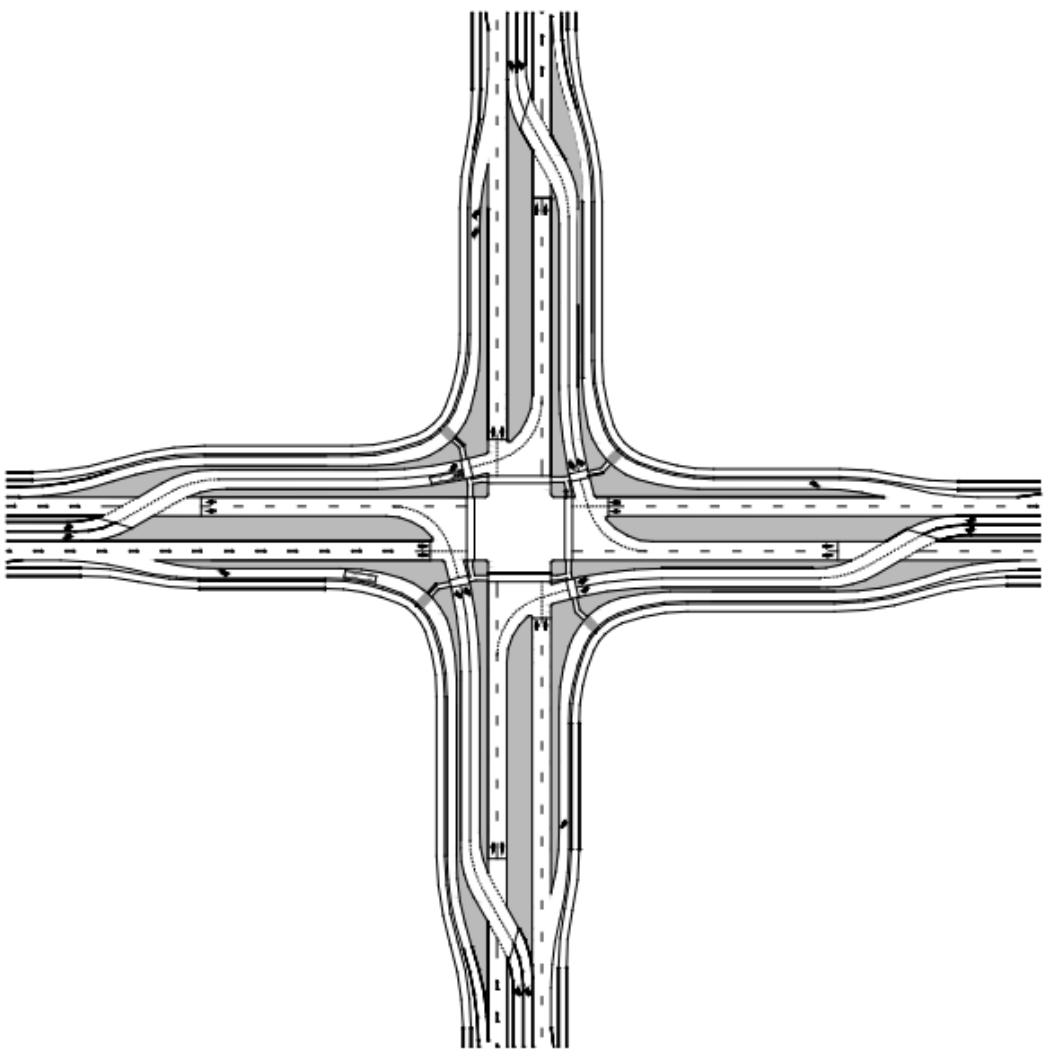


Figure 14-25: DLT Intersection with Displaced Left-Turns on All Approaches

Source: FHWA DLT Informational Guide

14.7.2 Design Considerations

The distance between the main intersection and the crossovers should generally range from 300-ft 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 14-26** for a depiction of typical intersection spacing.

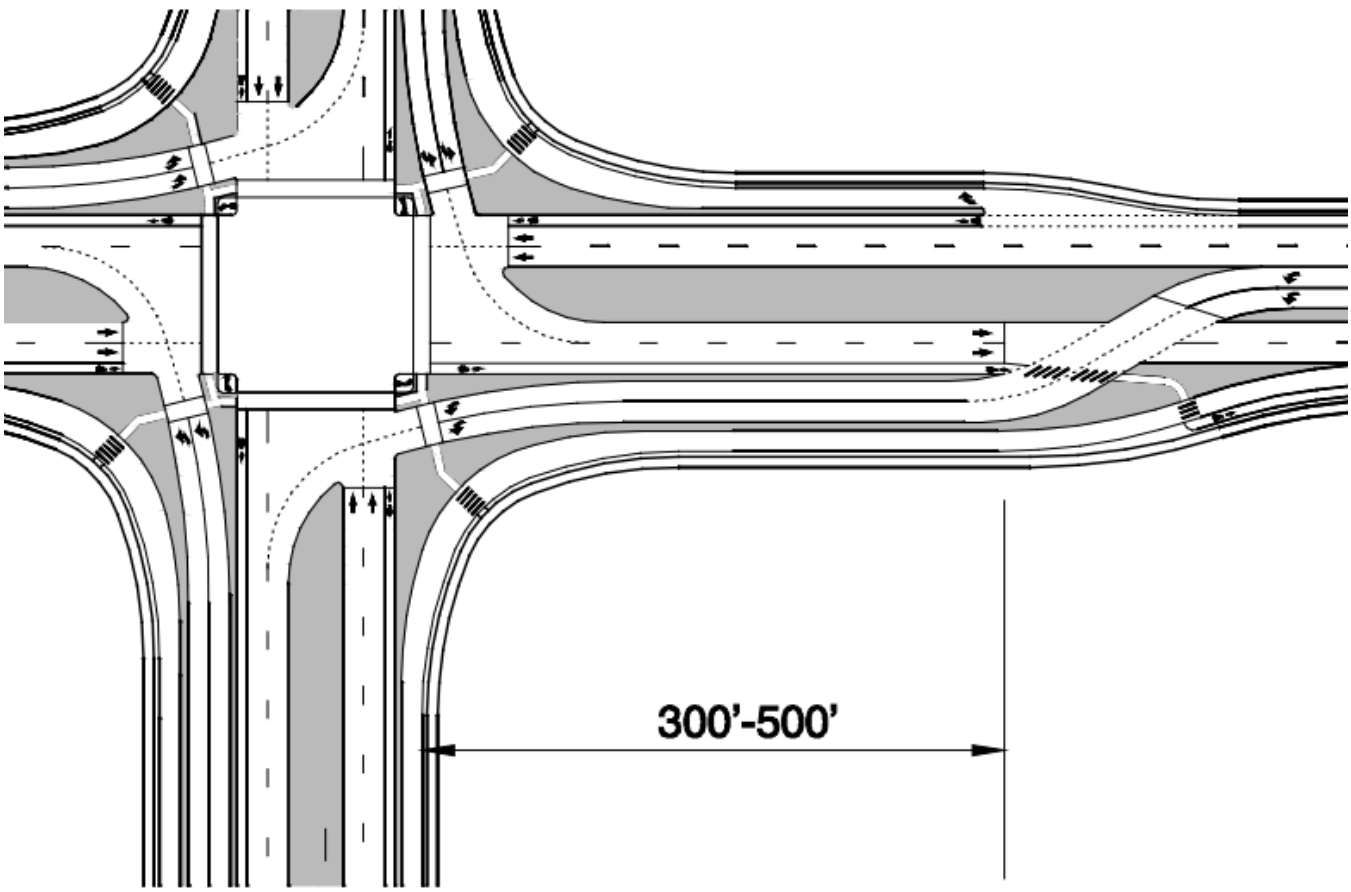


Figure 14-26: DLT Typical Intersection Spacing
Source: FHWA DLT Informational Guide

At the main intersection, the primary objective is to provide appropriate turning paths for the displaced left-turns, considering the interaction with sidewalks. Side-by-side check vehicles are not typically accommodated but can be if the context of the intersection has a high frequency use of check vehicles.

The vehicle paths for the displaced left-turns through the main intersection will delineate the curb lines and stop bar (a.k.a. stop line) locations and determine the width of the overall intersection as shown in **Figure 14-27**.

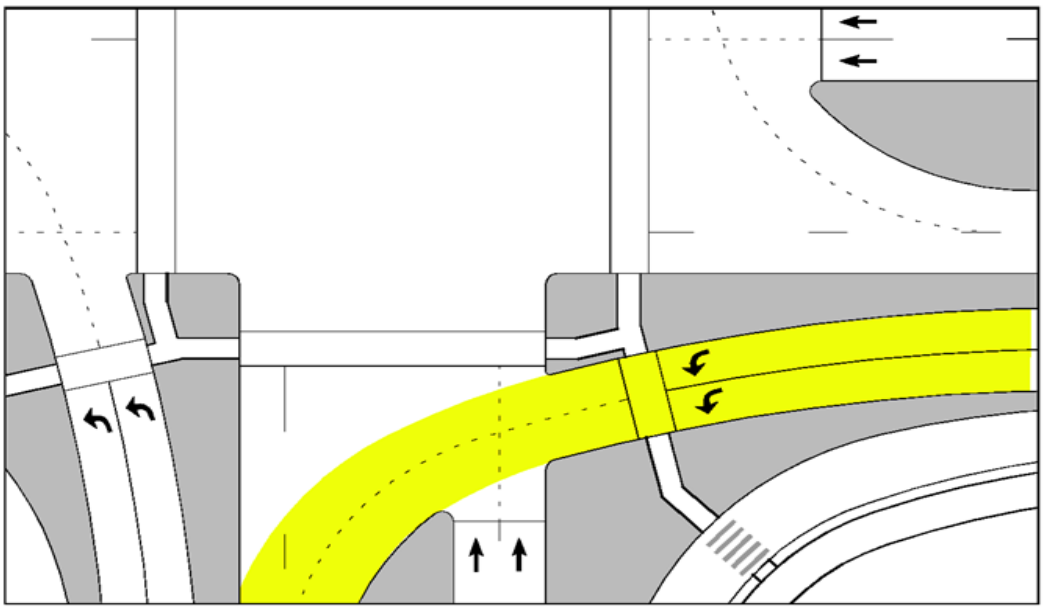


Figure 14-27: DLT Left-Turn Maneuvers
Source: FHWA DLT Informational Guide

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 14-28**.

It is desirable to provide the largest crossing angle while adapting to each site's unique conditions. Early design practice targeted crossover angles of 45 degrees or greater to facilitate proper vehicle path alignment through the crossover and reduce the likelihood of wrong-way movements. It is often difficult to achieve 45 degrees in practice. Several factors influence cross angle selection:

- **Wrong-way maneuvers** — Minimizing the likelihood of a wrong-way maneuver into opposing traffic is a key consideration. The greater the crossing angle, the more the intersection will appear conventional and support use of the crossover; and
- **ROW constraints** — The surrounding environment will influence a configuration. For instance, a retrofit design may be constrained by ROW.

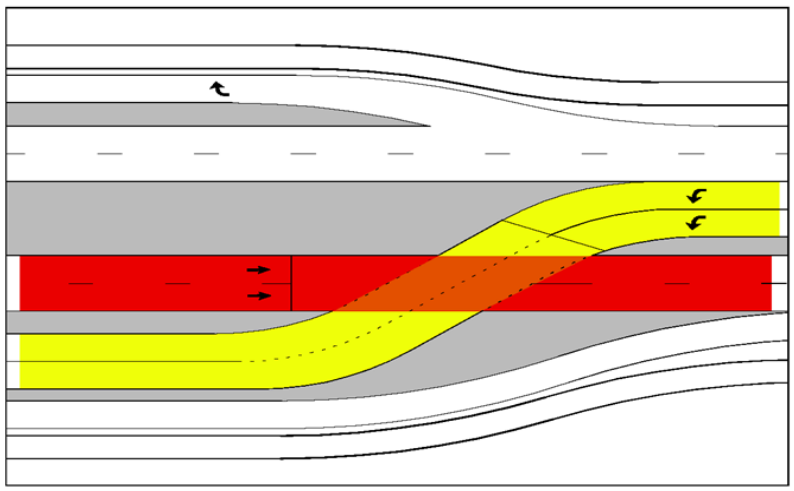


Figure 14-28: DLT Crossover Intersection Geometry
Source: FHWA DLT Informational Guide

Additionally, there are two ways to accommodate the geometry where the right-turn bypass lane joins the crossroad through lanes:

- Provide an additional lane with a downstream merge; or
- Signalize the movement and operate it as part of the crossover signal.

For more detailed information on the geometric design of a DLT intersection refer to Chapter 7 of the **FHWA DLT Intersection Informational Guide**, 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.

14.7.3 Pedestrian and Bicyclist Considerations

Crosswalks for DLTs are at the same locations that they would be for a conventional intersection. The major street crossing may be made in one or two stages, where the median can be used to provide a refuge for a two-stage crossing. **Figure 14-29** shows a typical pedestrian crossing with refuge islands.

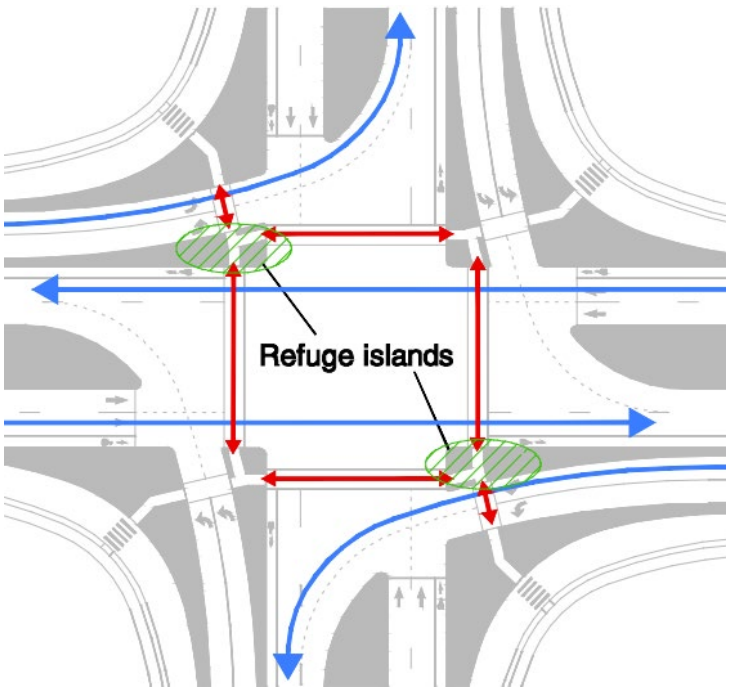


Figure 14-29: Refuge Islands Between Left-Turn and Through Lanes

Source: FHWA DLT Informational Guide

Various options exist for bicyclists to safely use a DLT. Where on-street bicycle lanes are provided on a corridor, they should continue through the DLT intersection. Marked "bike box" locations should be provided for left-turns. Since less experienced cyclists may not feel comfortable riding through the DLT, it is desirable to provide a ramp from the bike lane to a shared-use path which can be used by both pedestrians and cyclists. This is similar to the recommended practice at roundabouts.

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.

14.7.4 Access Management

Maintaining or providing access to homes and businesses near a DLT intersection can be accomplished by using frontage roads; however, weaving movements in and out of driveways or a need for U-turns at the main or adjacent intersections may result 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 configurations from the channelized right-turn lanes. Refer to the **NCHRP Report 420 Impacts of Access Management Techniques** for additional

guidance for access management for DLT intersections.

14.7.5 Safety

DLT intersections provide safety benefits due to the reduction of conflict points within the intersection. Safety issues to consider with displaced left-turn intersections include:

- Pedestrian and bicycle conflicts;
- Potential for wrong-way movements; and
- Potential impact on emergency vehicle response times in the event that the intersection is blocked or there has been a loss of power to the signal.

For additional information on safety, refer to the **FHWA Displaced Left-Turn Intersection Informational Guide**.

14.7.6 Operations

DLT intersections can improve traffic flow by separating left-turning vehicles from through traffic. The DLT is a system of multiple signalized intersections. Signal timing and phasing are critical to the performance of a DLT intersection. Right-turns on red are prohibited as shown in **Figure 14-30**.

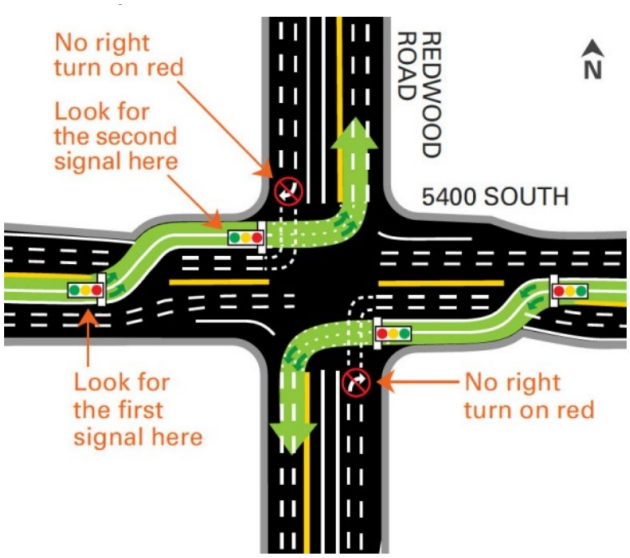


Figure 14-30: Example Continuous Flow Intersection (DLT)

Source: TxDOT

14.7.7 Traffic Control Devices

DLT intersections require clear traffic control devices to guide drivers through the intersection. The main objectives of traffic control devices in DLT intersections are:

- Position signal heads above the receiving lanes at the crossovers;
- Provide signage and pavement markings to prohibit wrong-way movements;
- Provide lane extension striping through the main crossover intersection; and

- Provide pavement markings and overhead signs as supplemental guidance.

Refer to the **FHWA Displaced Left-Turn Intersection Informational Guide** for more information on traffic control devices at DLTs. The **TMUTCD** remains the primary source for traffic control device standards.

14.8 Median U-Turn Intersection (MUT)

14.8.1 Overview

A Median U-Turn (MUT) or "Michigan Left" intersection 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 Intersection Informational Guide** as the primary source for MUT Guidelines. The information contained in this section is considered a companion guide and is intended to document TxDOT's suggested approach to MUT design. See **Figure 14-31** for an example MUT configuration.

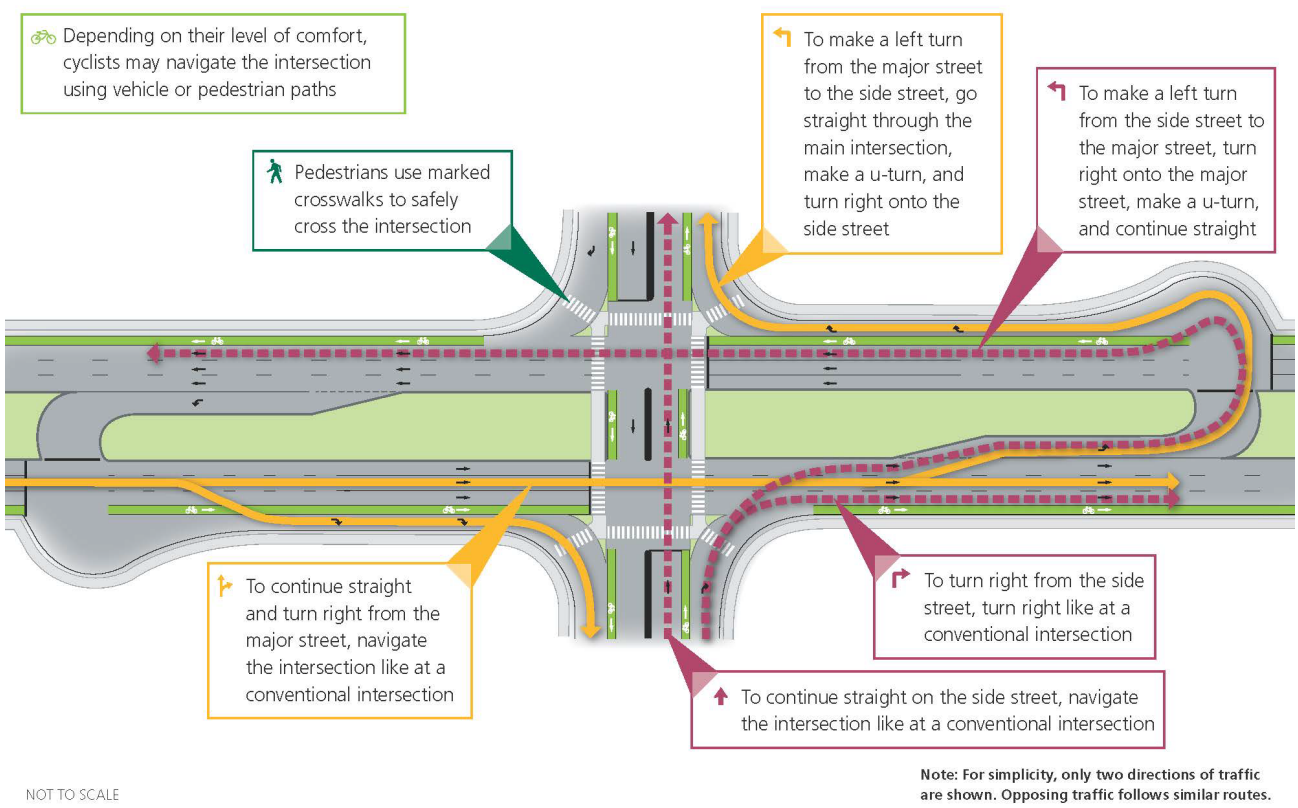


Figure 14-31: Example of a MUT Intersection With One Signal in Main Intersection

Source: Virginia DOT, used by permission

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; and
- General increase in safety performance.

Conditions where a MUT may not be suitable include:

- **High traffic volumes** – the addition of U-turn movements can cause spillback into the through lanes and increase congestion;
- **Narrow medians** – this can make it difficult for larger vehicles to maneuver the U-turn and may require turning loons; and
- **Inadequate driver familiarity** – if drivers do not understand how to navigate the intersection it may result in crashes.

14.8.2 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-ft to 600-ft, although this distance may be modified per guidance in the **FHWA Median U-Turn Intersection 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 **Figure 14-32** through **Figure 14-34** for spacing considerations.

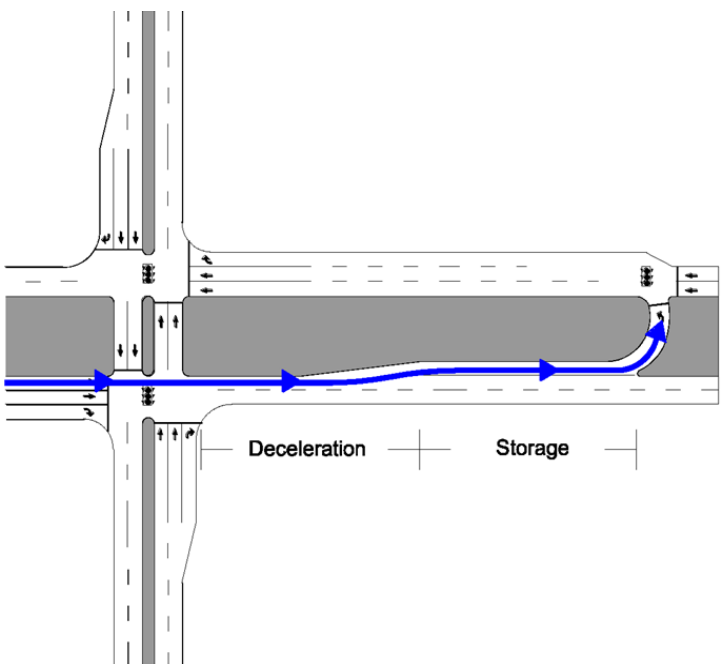


Figure 14-32: Spacing Consideration for a Major Street Left-Turn Movement

Source: FHWA MUT Informational Guide

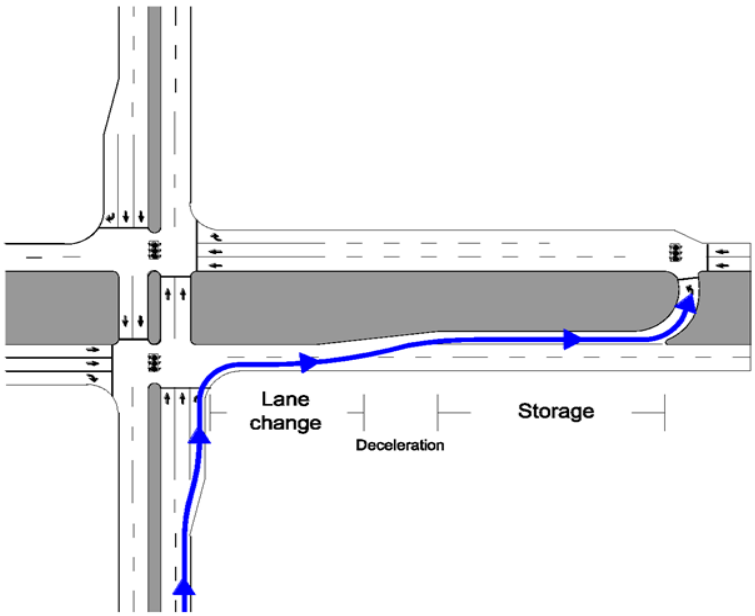


Figure 14-33: Spacing Consideration for Minor Street Left-Turn Movement

Source: FHWA MUT Informational Guide

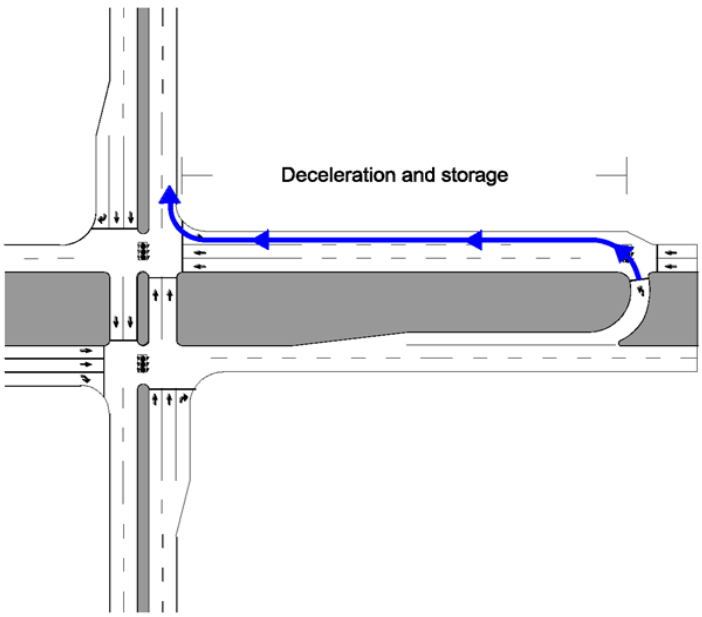


Figure 14-34: Spacing consideration for a Right-turn

Source: FHWA MUT Informational Guide

The MUT median width is a function of the design vehicle (typically a WB-62TX or WB-67) and the preferred alignment of the design vehicle as it turns into the opposing traffic. See **Chapter 13** 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 14-35** as an example application for a longer weaving distance.

14.8.3 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. Pedestrian crossings may be either single-stage (walk signal is timed to allow crossing the entire roadway on a single walk indication) or two-stage (a separate walk signal is used to cross each direction of a divided roadway).

Additional pedestrian crossings may be installed at U-turn crossover locations. With vehicular left-turn lanes removed from the crossing locations, the lengths of crossings for pedestrians can often be reduced compared to a conventional intersection.



Figure 14-35: Pedestrian Crossings at U-turn Crossover Location
Source: NCHRP Report 948 for Pedestrian and Bicycle Safety at Alternative and Other Intersection and Interchanges

Bicyclists have three options for navigating a MUT intersection when making a left-turn:

- **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.

- **Bicyclists follow pedestrian crossing rules (potential option):** Bicyclists approach the intersection and exit the street to the right and follow the pedestrian ("walk"/"don't walk") indications.
- **Bicyclists following vehicle rules (legal but undesirable option):** 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 14-36 shows the three bicyclist options.

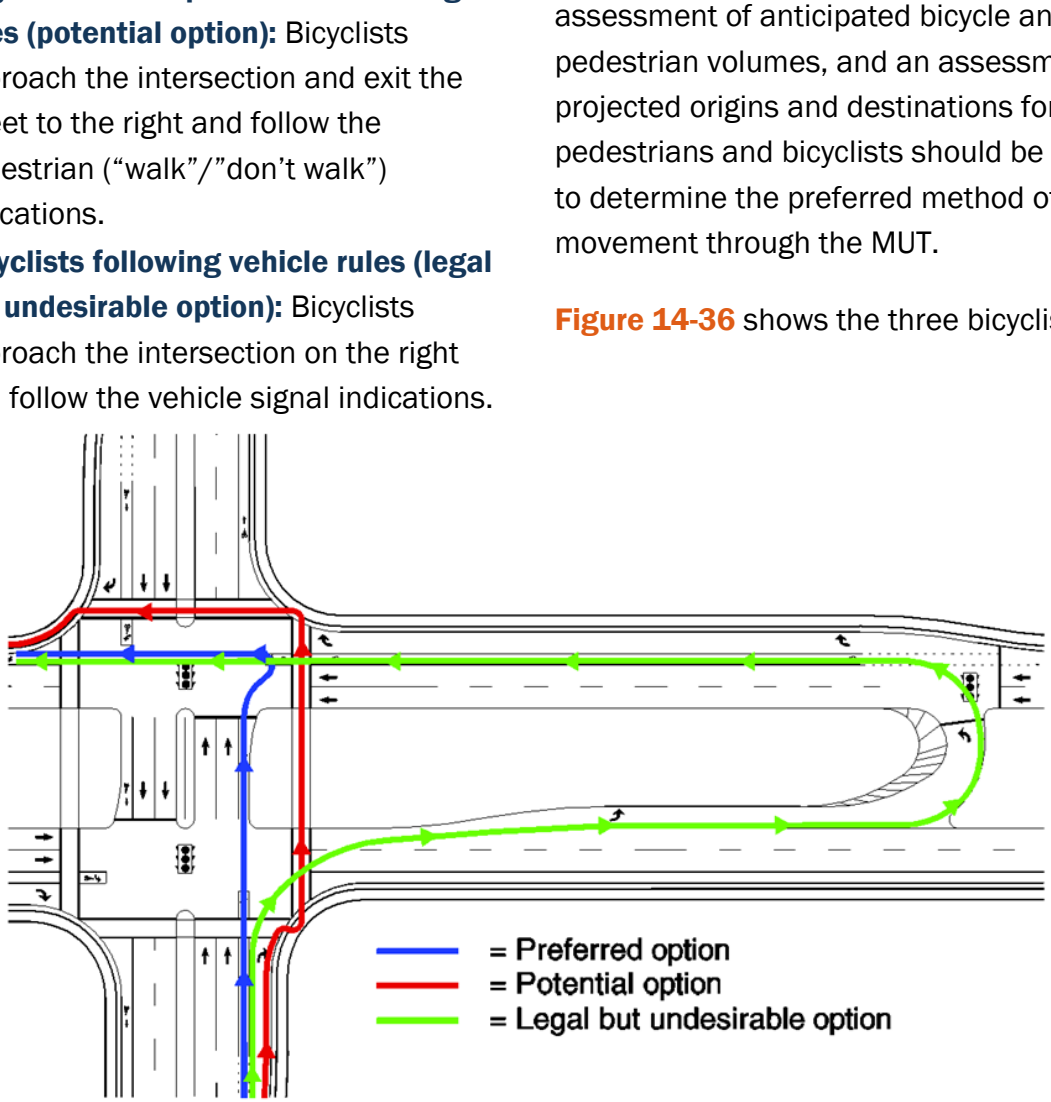


Figure 14-36: Left-Turn Options for Bicycles
Source: FHWA MUT Informational Guide

14.8.4 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. Refer to the **FHWA Alternative Intersections/Interchanges Informational Report (AIIR)** for additional guidance with respect to access management.

Additionally, corridor wide access strategies and management considerations to properties along the major street should be considered to promote safe and efficient access to these properties. Refer to **NCHRP Report 948**.

14.8.5 Safety

MUT intersections have safety benefits due to reduced conflict points, especially crossing conflict points, resulting in fewer severe crashes. Potential safety issues to consider at median U-turn intersections include:

- Conflicts where U-turns are aligned with driveways or streets that only permit right-turns;
- The need for channelization to prevent wrong-way movements; and
- The potential for weaving movements on the major street.

For additional information on safety refer to the **FHWA Median U-Turn Intersection Informational Guide**.

14.8.6 Operations

The MUT intersection provides operational benefits, especially for the through movement on the major street. Signal timing and phasing for the MUT and corridor are critical to the operational performance of MUT intersections. Refer to the **FHWA Median U-Turn Intersection Information Guide** for further operational considerations.

14.8.7 Traffic Control Devices

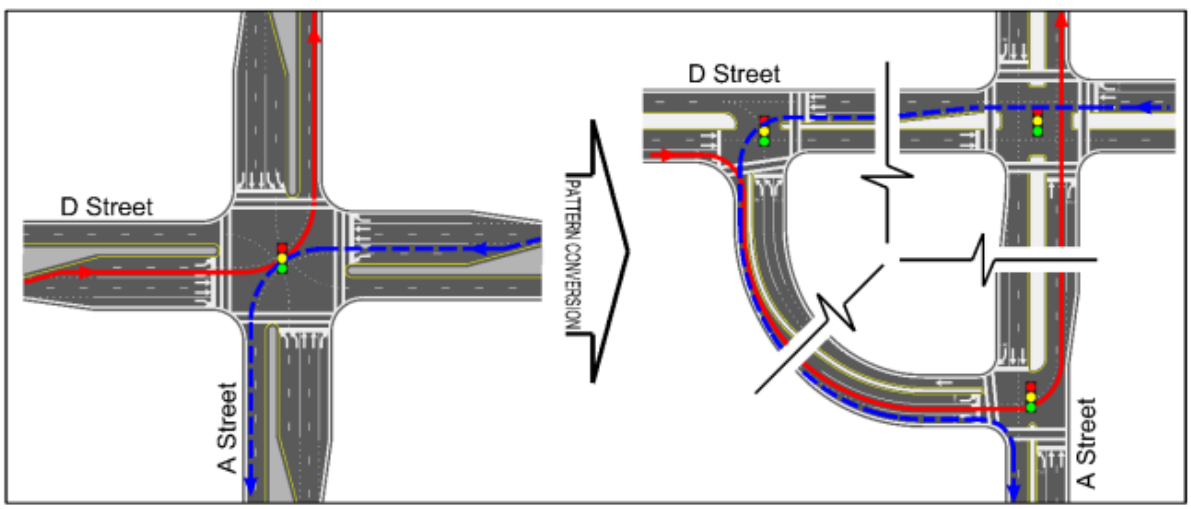
MUT intersections require clear traffic control devices to guide drivers through the intersection. The four main objectives of traffic control devices in MUT intersections are:

- Provide signage and pavement markings to prohibit left-turns and alternative routing of left-turn movements;
- Provide one-way and wrong-way signage;
- Provide a means for direct or indirect bicycle left-turn movements; and
- Provide appropriate safety lighting at conflict points.

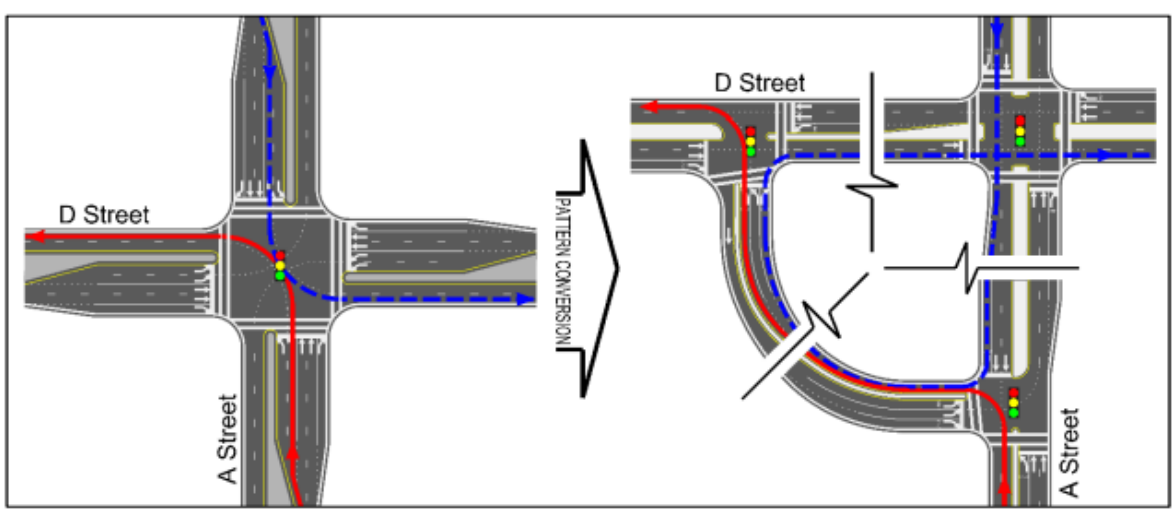
Refer to the **FHWA Median U-Turn Intersection Information Guide** for further guidance on traffic control devices. The **TMUTCD** remains the primary source for traffic control device standards.

14.9 Quadrant Roadway Intersections

A quadrant roadway (QR) intersection displaces left-turns to a quarter-arc "quadrant roadway" in one of the intersection's quadrants as shown in **Figure 14-37**. The primary benefit of a QR intersection is a reduction in traffic signal phases and conflict points, resulting in improved intersection operations. More information on QR intersections can be found in the **Quadrant Roadway Intersection Informational Guide, January 2020**, published by the FHWA.



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Figure 14-37: Quadrant Roadway Intersection Example

Source: Quadrant Roadway Intersection Informational Guide, January 2020

14.10 Diverging Diamond Interchange (DDI)

14.10.1 Overview

The Diverging Diamond Interchange (DDI) is an interchange form where the two directions of traffic on the crossroad temporarily divide and cross to the opposite side to gain access to and from the freeway frontage roads more efficiently. 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 crossovers, vehicles on the crossroad making a left-turn on to or off of ramps, or frontage roads, do not conflict with vehicles approaching from other directions. TxDOT has adopted the **NCHRP Report 959 Diverging Diamond Interchange Informational Guide** as the primary source for DDI guidelines. **Figure 14-38** depicts a schematic concept of the DDI Interchange configuration.

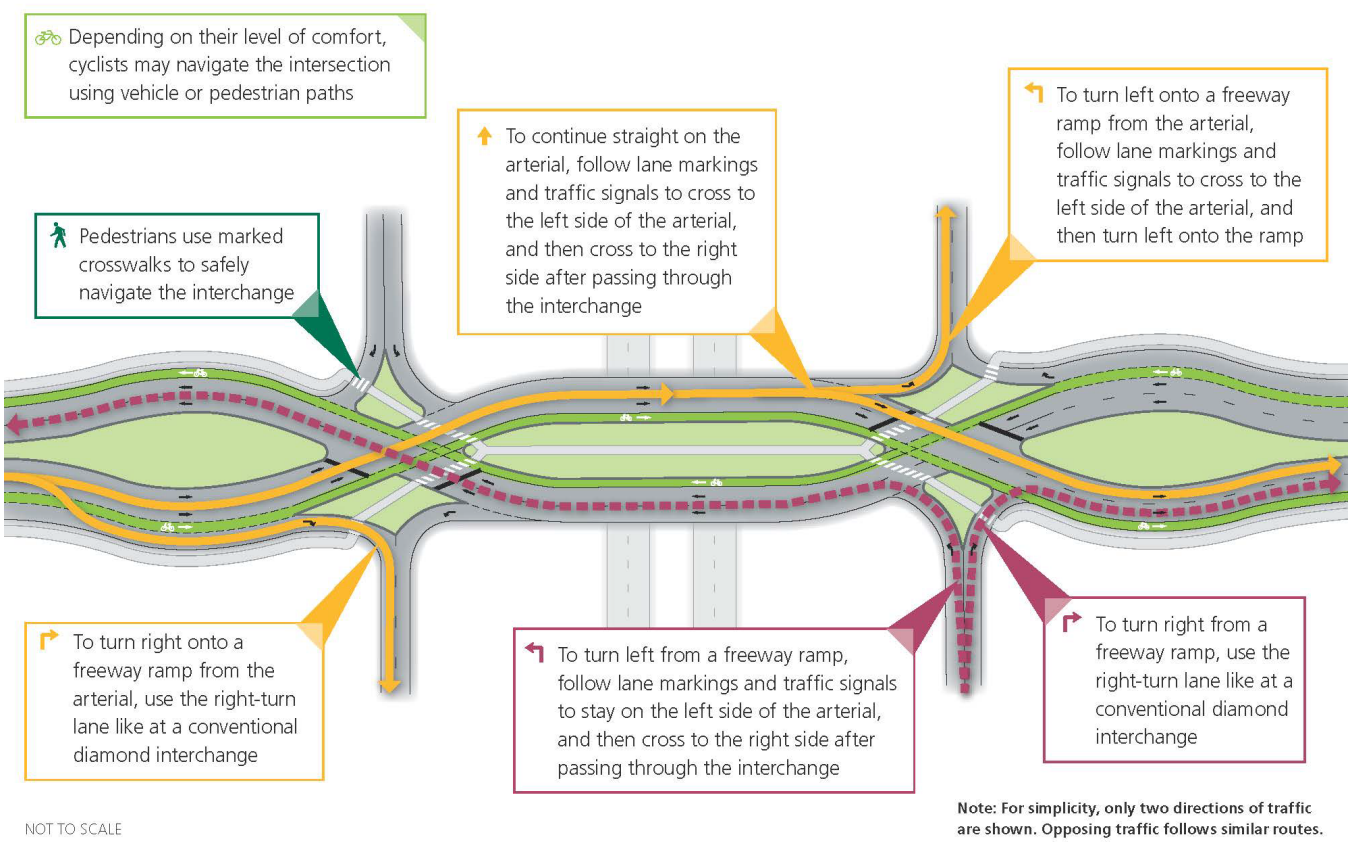


Figure 14-38: Diverging Diamond Interchange Concpet (DDI)
Source: Virginia DOT, used by permission

The information contained in this section is considered a companion to the FHWA Guide and is intended to document TxDOT's suggested approach to DDI design.

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; and
- Possible lower costs due to a smaller footprint, shorter construction time, and the possibility of being able to salvage existing bridge structures.

Conditions where a DDI may not be suitable include:

- **Densely developed areas** – DDI's can restrict access to existing developments due to the crossovers near the interchange; and
- **Inadequate driver familiarity** – DDI's require public involvement and education as well as significant signage to assist drivers navigating the interchange.

Figure 14-39 shows the design characteristics and key features of the DDI.

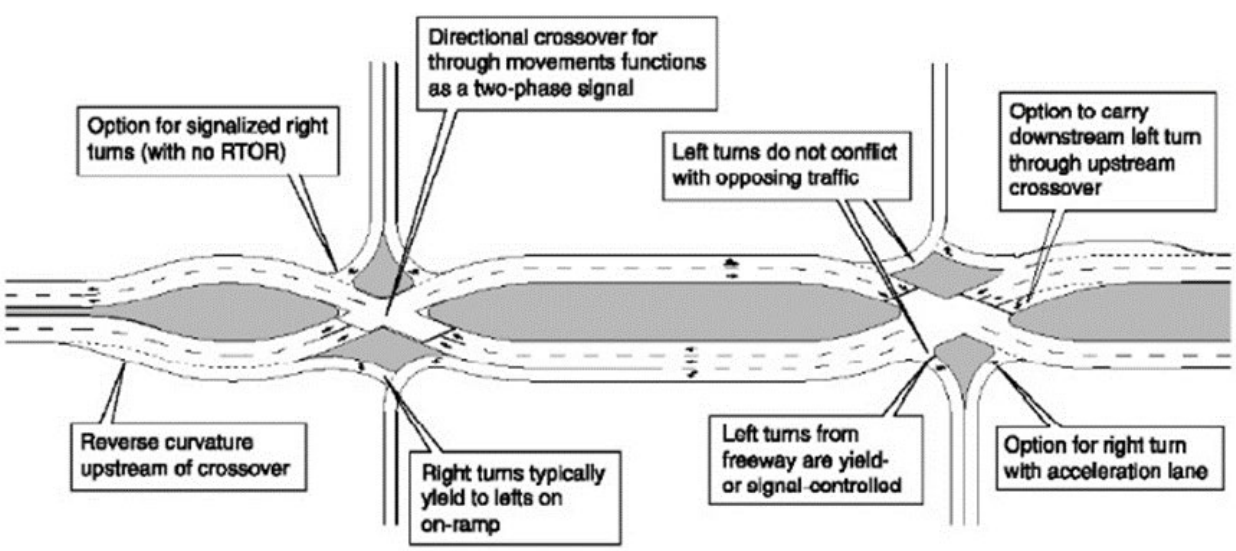


Figure 14-39: Key Characteristics of a DDI

Source: FHWA DDI Informational Guide

14.10.2 Design Considerations

Development of functional geometrics are central to safe and efficient operation of a DDI. The most common design vehicle used at DDI facilities is the WB-67. In more urbanized areas, it may be appropriate to use smaller design vehicles, such as the WB-62TX or WB-40. The anticipated truck movements, staying

in-lane through the DDI, should be determined and AutoTURN (or a similar tool) used to assess vehicle encroachments within the DDI. 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 (**Table 4-5** and **Table 4-6** 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 **Figure 14-40** and **Figure 14-41**). A consolidated ramp terminal intersection with crossover is desired over separated ramp terminal intersection elements (see **Figure 14-42**).

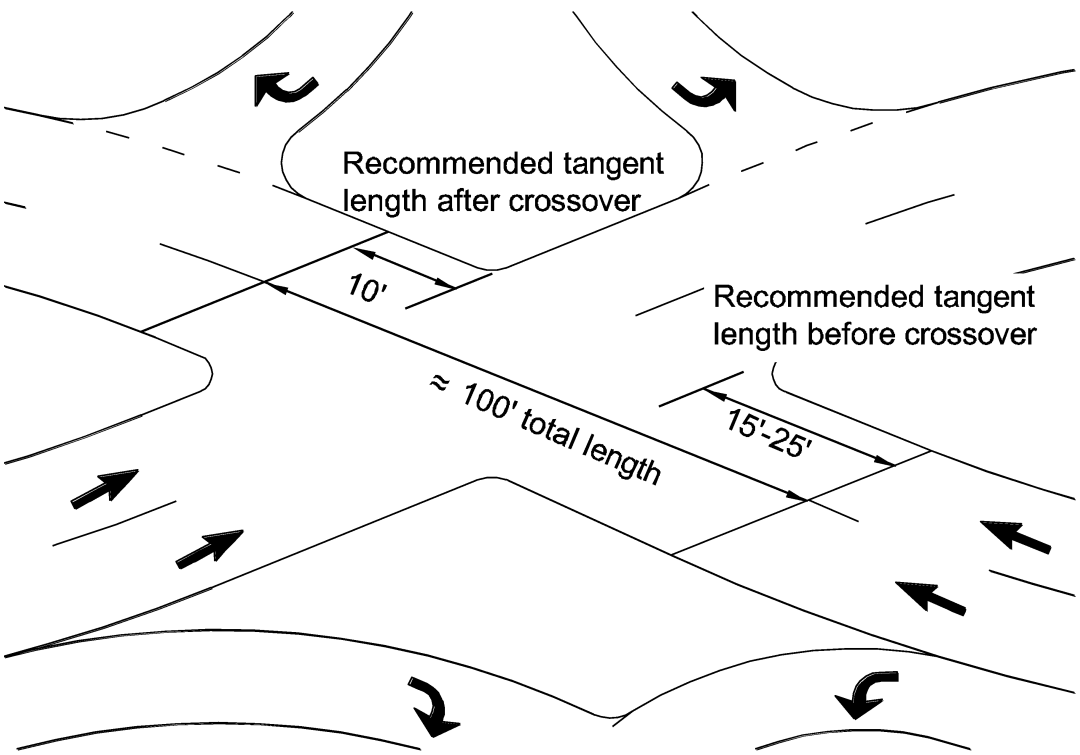


Figure 14-40: Tangent Length Approaching and Departing the Crossover

Source: FHWA DDI Informational Guide

Curve radii approaching and following the DDI crossover generally range from 150-ft to 300-ft. At the crossover, the greater the crossing angle, the less "different" the intersection will seem. FHWA's Intersection and Interchange Geometrics (IIG) Workshop, May 2016, recommends a crossover angle of 40 to 50 degrees. The 2016 IIG Workshop also noted the following regarding low crossover angles:

- Low crossover angles may increase the likelihood for wrong-way maneuvers into the opposing lanes; and
- Low angles increase distances and increase signal clearance time.

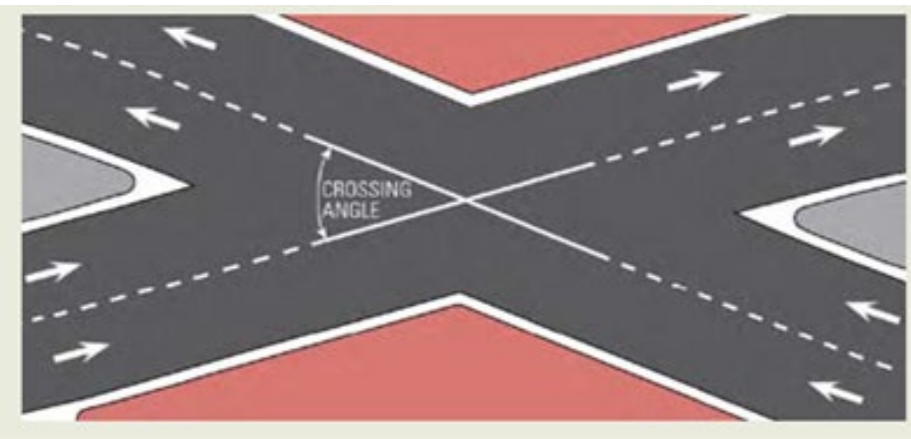


Figure 14-41: DDI Crossover Geometry (Crossing Angle)

Source: FHWA Intersection and Interchange Geometrics (IIG) Workshop (May 2016)

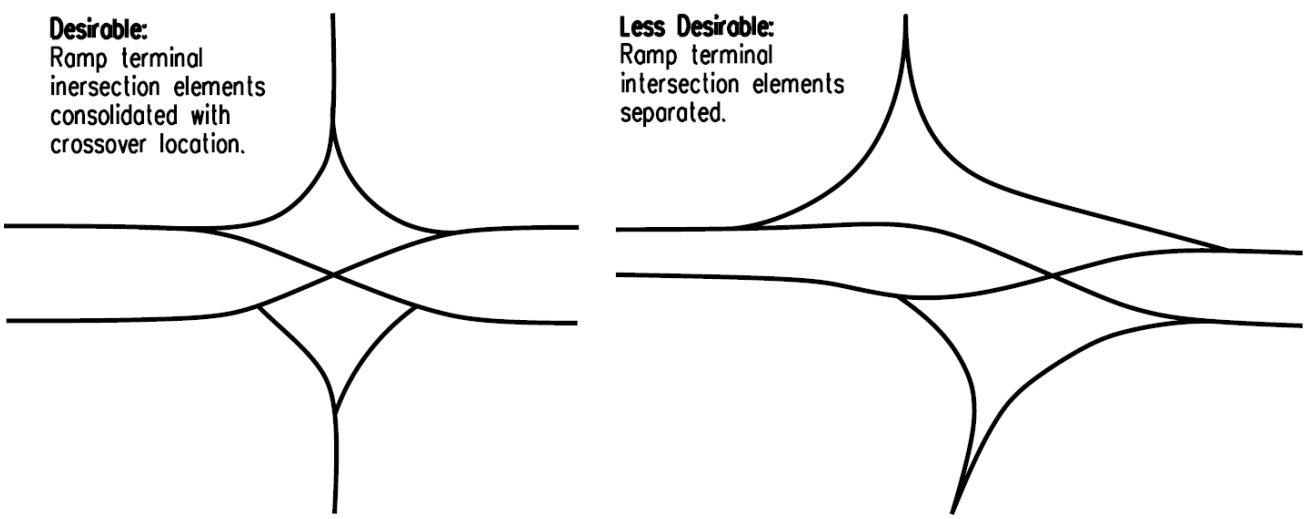


Figure 14-42: Consolidating the Ramp Terminal Intersection

Source: FHWA DDI Informational Guide

14.10.3 Sight Distance

Drivers approaching or departing an intersection must have an unobstructed view of traffic control devices and sufficient length along the crossroad 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.

Due to their curvilinear alignment, DDIs have special considerations that need to be reviewed and coordinated closely with the horizontal alignment. Primary considerations for vertical alignment at DDIs include:

- **Crest vertical curve on the overcrossing** - A crest vertical curve on an overcrossing may potentially mask the turning roadway alignment to the crossover;
- **Skew in an overcrossing** - A crest vertical curve and skew angle can also result in reduced visibility to the crosswalk across the left-turning roadway; and
- **Crosswalk visibility** - The relatively sharp horizontal curve to the left and bridge parapet can obscure views to a user waiting at the pedestrian crossing.

Refer to **NCHRP Report 959** for additional information regarding sight distance considerations.

14.10.4 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 14-43** shows an example of symmetrical and shifted alignments. This results in distances of 600-ft to 750-ft between crossovers. If the distance between crossovers can be reduced, it can boost traffic operations and limit the amount of ROW needed.

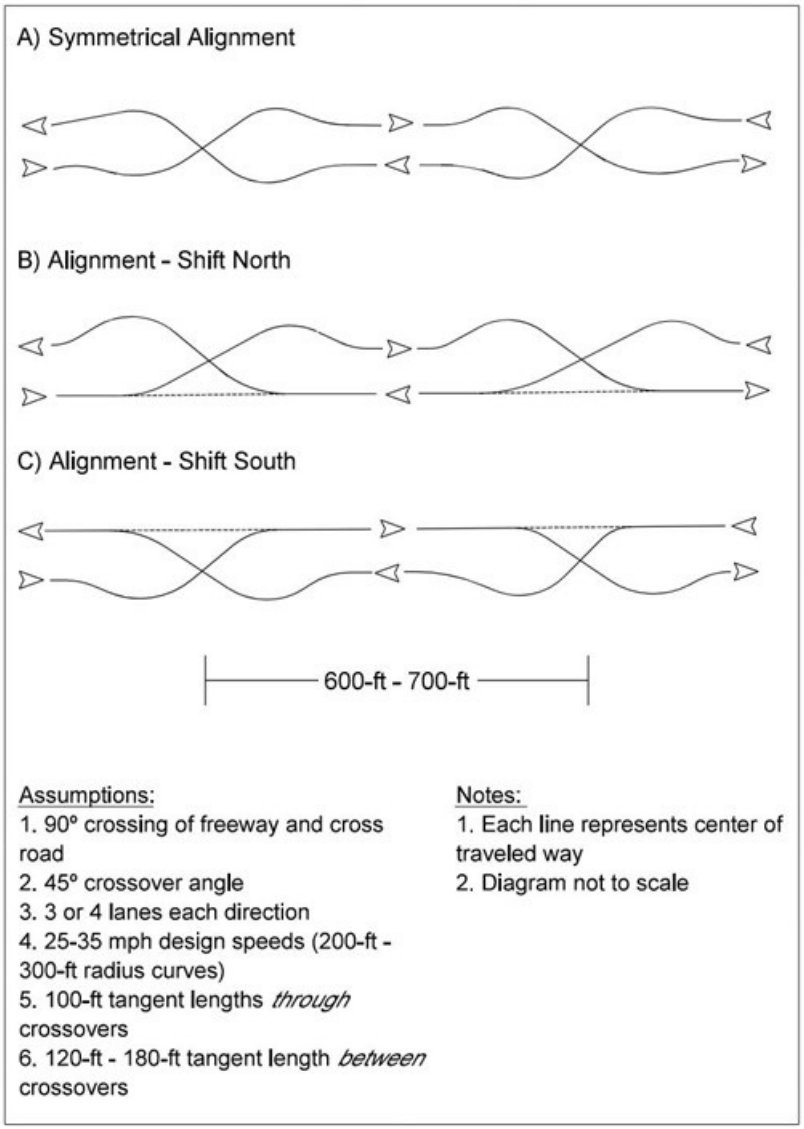


Figure 14-43: Alignment Alternatives

Eliminating a few reverse curves can reduce the spacing between crossovers. **Figure 14-44** 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-ft to 500-ft between crossovers. Chapter 7 of **NCHRP Report 959** provides additional information on horizontal alignment options.

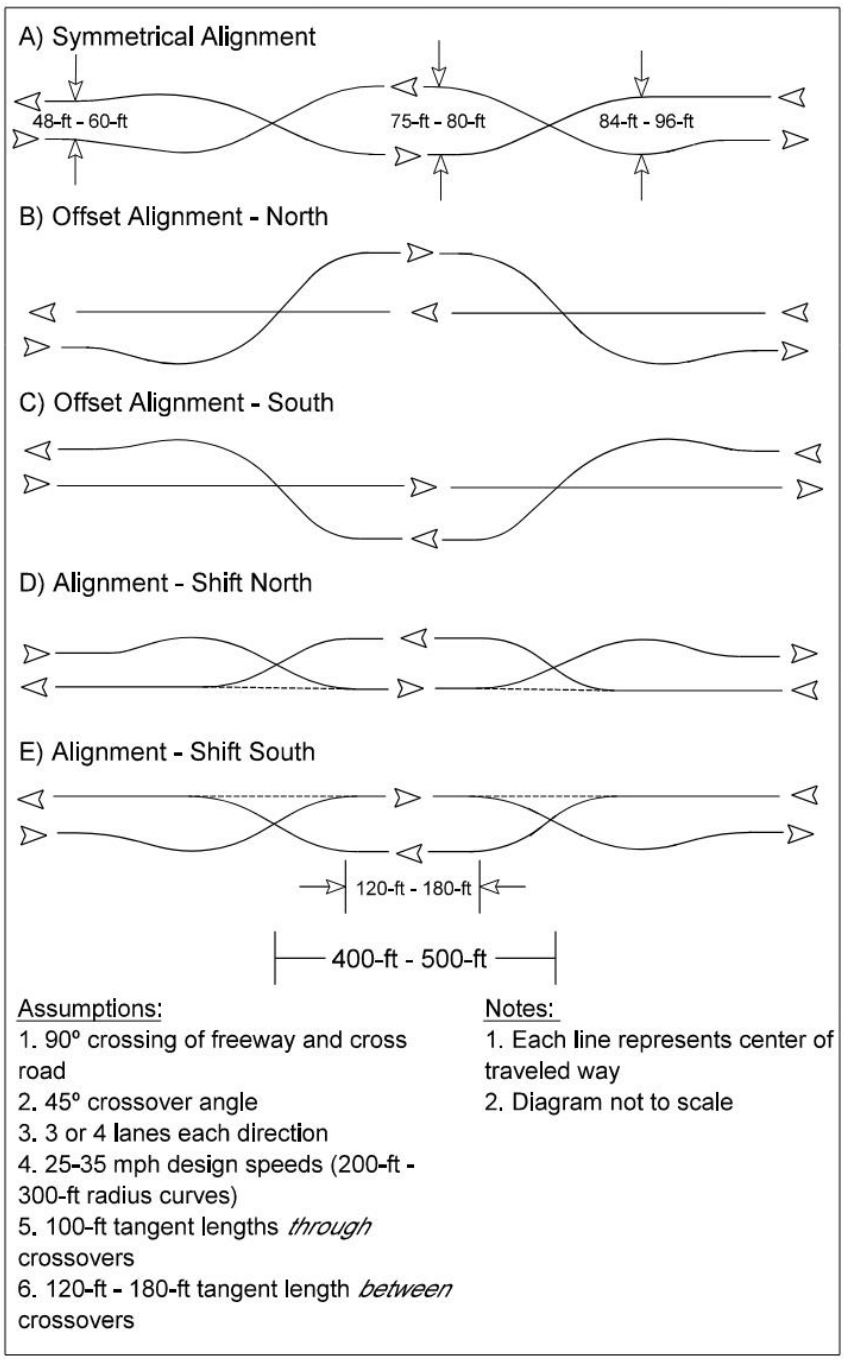


Figure 14-44: Reduced Reverse Curves in Alignment Alternatives

14.10.5 Auxiliary Lanes

Auxiliary lanes can aid weaving traffic at a DDI interchange, smooth traffic flow, provide added capacity, and improve overall safety. **Figure 14-45** through **Figure 14-48** show examples of auxiliary lanes at a DDI interchange. Chapter 7 of **NCHRP Report 959** provides additional information on auxiliary lanes.

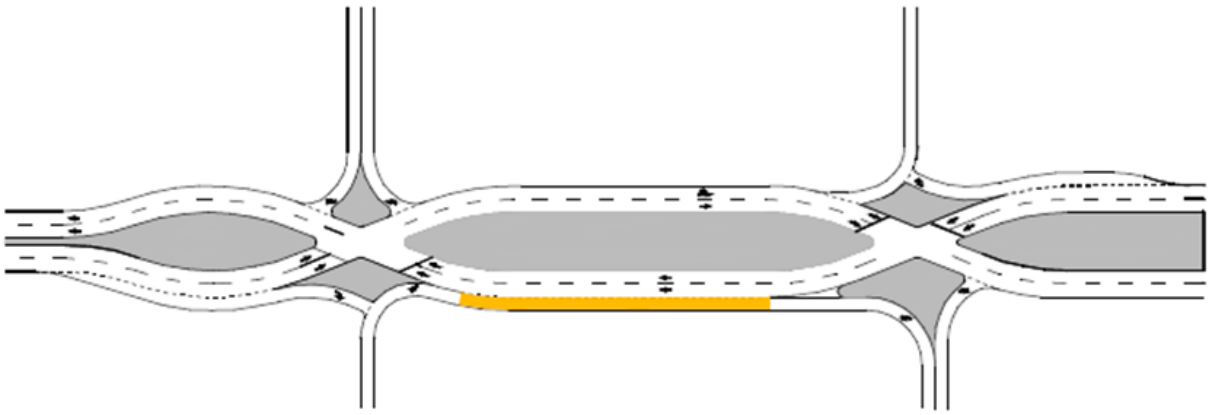


Figure 14-45: Auxiliary Left-Turn Lane Between Crossovers (Not Preferred)

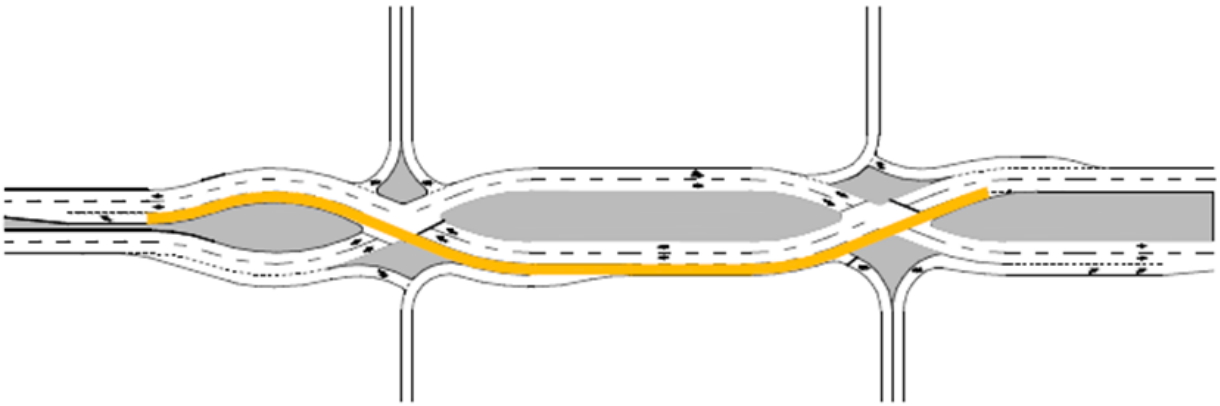


Figure 14-46: Auxiliary Left-Turn Lane Developed Prior to the First Crossover

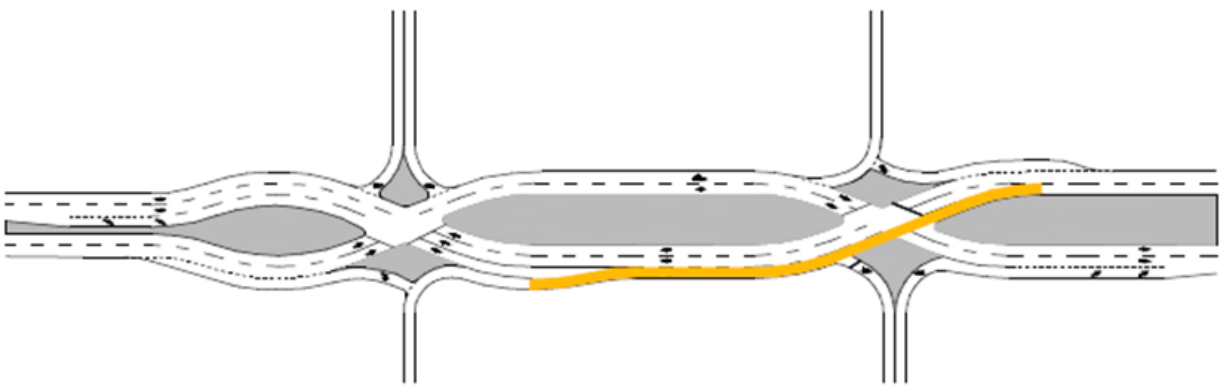


Figure 14-47: Auxiliary Lane, Shared Left and Through, Developed Prior to the First Crossover

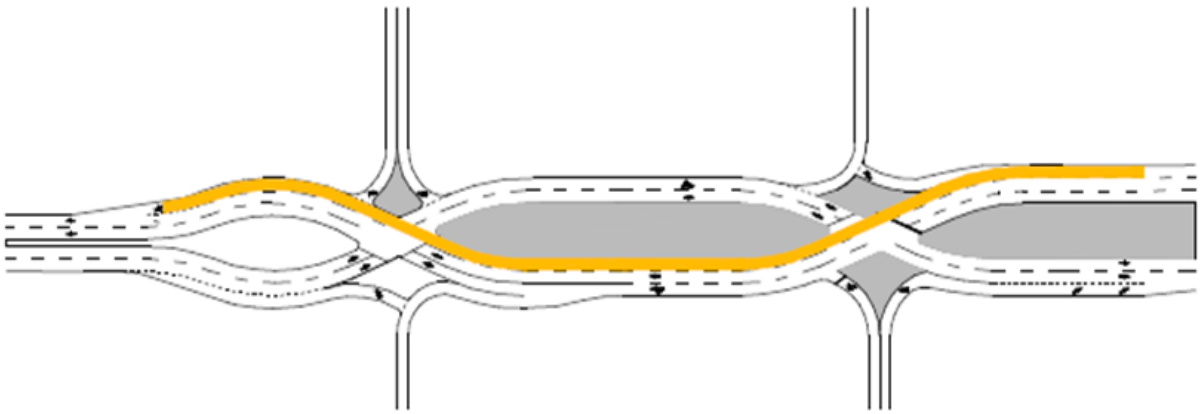


Figure 14-48: Auxiliary Through Lane Developed Prior to the First Crossover

14.10.6 Ramp Terminals

Ramp Terminal geometry is discussed in **NCHRP Report 959**, Section 6.4.8 and is shown in **Figure 14-49**.

The contemporary configuration shown in **Figure 14-49** is considered to be the safer solution for DDI ramp terminal geometry based on the enhanced reduction in vehicle speed while offering greater view angles that require drivers to turn their heads to less of a degree while trying to identify oncoming traffic.

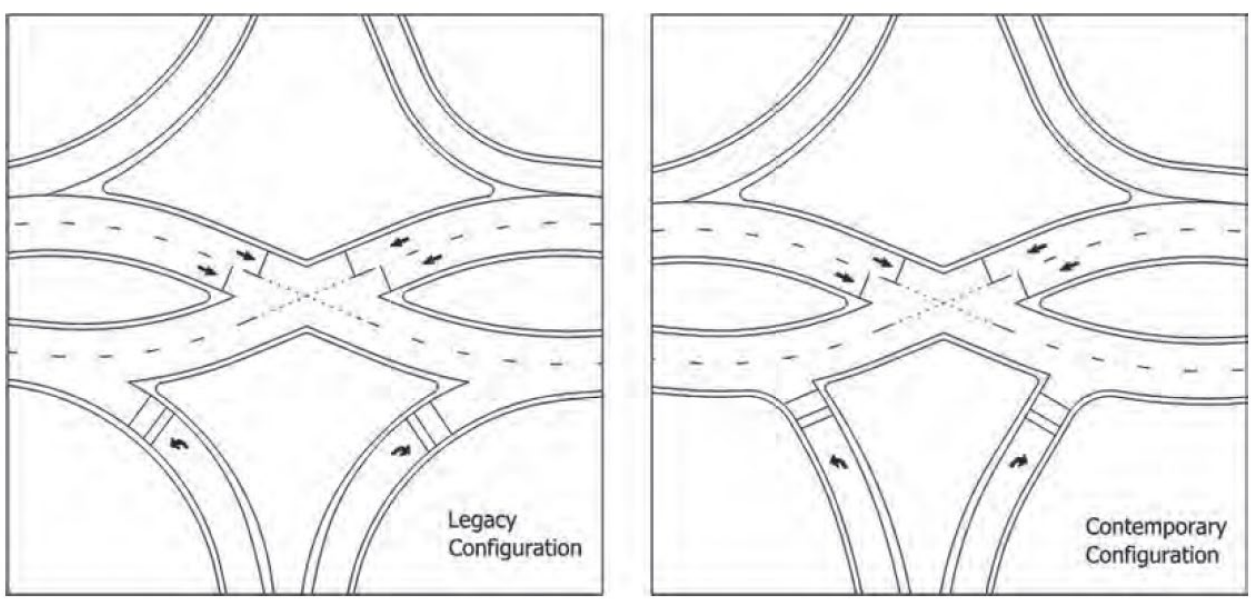


Figure 14-49: Ramp Terminal Geometry at a DDI

14.10.7 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 for additional protection and channelization for 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, 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 designing the turning radii to reduce speeds in the vicinity of pedestrians. **Figure 14-50** and **Figure 14-51** depict the inside and outside pedestrian sidewalk options, respectively.

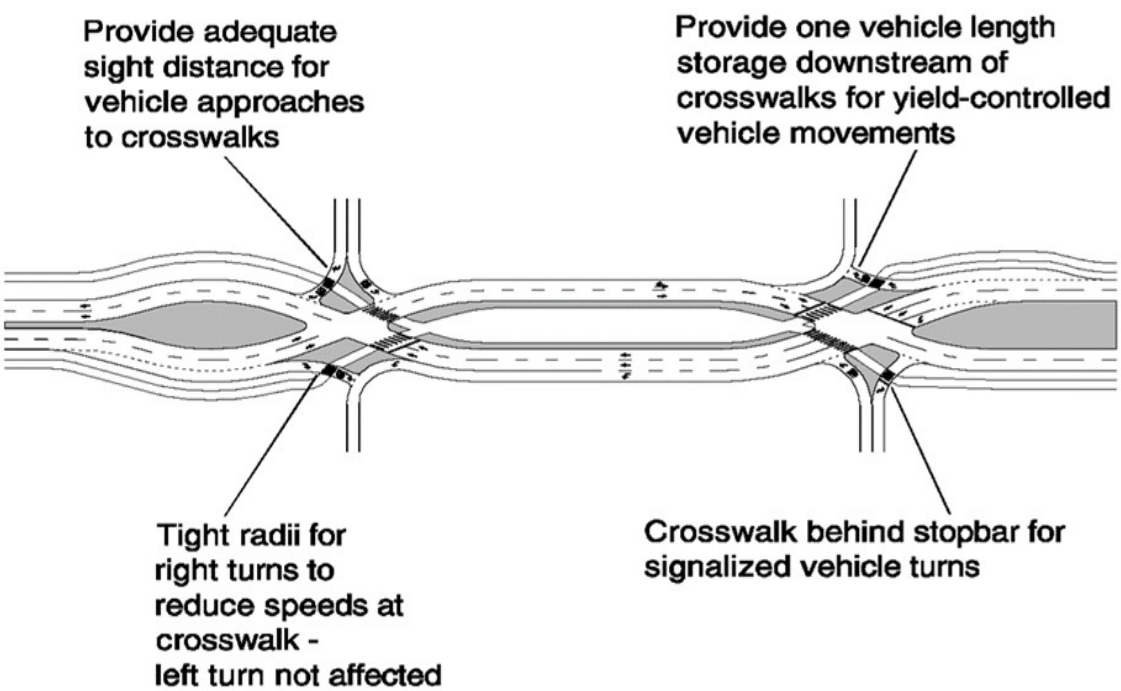


Figure 14-50: Pedestrian-focused DDI – Center Walkway

Source: FHWA DDI Informational Guide

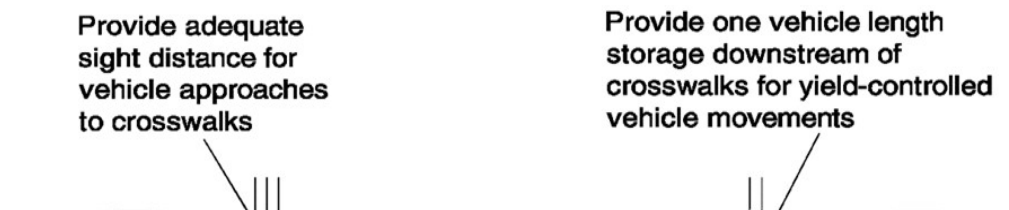


Figure 14-51: Pedestrian-focused DDI – Outside Walkway

Source: FHWA DDI Informational Guide

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-ft to 7-ft are recommended through the DDI. See **Figure 14-52**; and
- **A separated sidewalk or wider shared-use path** - This would typically entail the bicyclist 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 14-52: Schematic for bicycle lane placement on right side of vehicular traffic

Source: FHWA DDI Informational Guide

14.10.8 Access Management

DDI intersections require 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. 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 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; and
- Removes driveway access for corner development.

Refer to the **FHWA Alternative Intersections/Interchanges Informational Report (AIIR)** for additional guidance with respect to access management.

14.10.9 Safety

DDIs have safety benefits due to reduced conflict points, especially crossing conflict points, resulting in fewer severe crashes. They also decrease the likelihood of wrong-way maneuvers onto the intersecting expressway. Caution should be exercised when allowing

right-turn-on-red and left-turn-on-red maneuvers, as drivers may look to the wrong side of the road for oncoming traffic. For additional information on safety, refer to the **NCHRP Report 959**.

14.10.10 Operations

DDI operations may vary slightly based on location and traffic demand. There are three potential phasing schemes:

- Two-critical movement phasing;
- Three-critical movement phasing; and
- Four critical movement phasing.

Signal timing and phasing of the adjacent intersections must be coordinated to provide appropriate capacity to the DDI. Refer to the **NCHRP Report 959** for more information on operations.

14.10.11 Traffic Control Devices

DDIs require clear traffic control devices to guide drivers through the interchange. The four main objectives of traffic control devices in DDIs are:

- **Priority** – describes which movement has the ROW;
- **Directional guidance** – informs drivers about upcoming turning movements;
- **Lane choice** – communicates to drivers which lane to choose prior to entering the DDI; and
- **Information** – about other aspects of the interchange. Care must be used when providing informational signs to avoid distracting drivers.

Refer to **NCHRP Report 959** for more information on traffic control devices. The **TMUTCD** remains the primary source for traffic control device standards.

14.10.12 Illumination

At a minimum, safety lighting should be provided for DDIs in rural locations. Complete illumination systems should be considered at all DDIs to enhance visibility and safety. Refer to TxDOT's **Highway Illumination Manual** for additional information and design standards.

14.11 Contraflow Left Interchange (CLI)

Contraflow Left Interchanges (CLI) are a grade-separated interchange type that separate the left-turn movements of an interchange so that traffic from multiple approaches can make left-turn movements onto a freeway during the same signal phase. CLIs use channelized lanes for left-turns to allow a higher volume of left-turn traffic through the interchange in one signal phase. This interchange type increases traffic flow through the intersection and reduces the likelihood of congestion and back-ups that cause rear-end crashes. See **Figure 14-53** for an illustration of a Contraflow Left Interchange. Refer to **Innovative Intersections webpage** on TxDOT.gov for additional information.

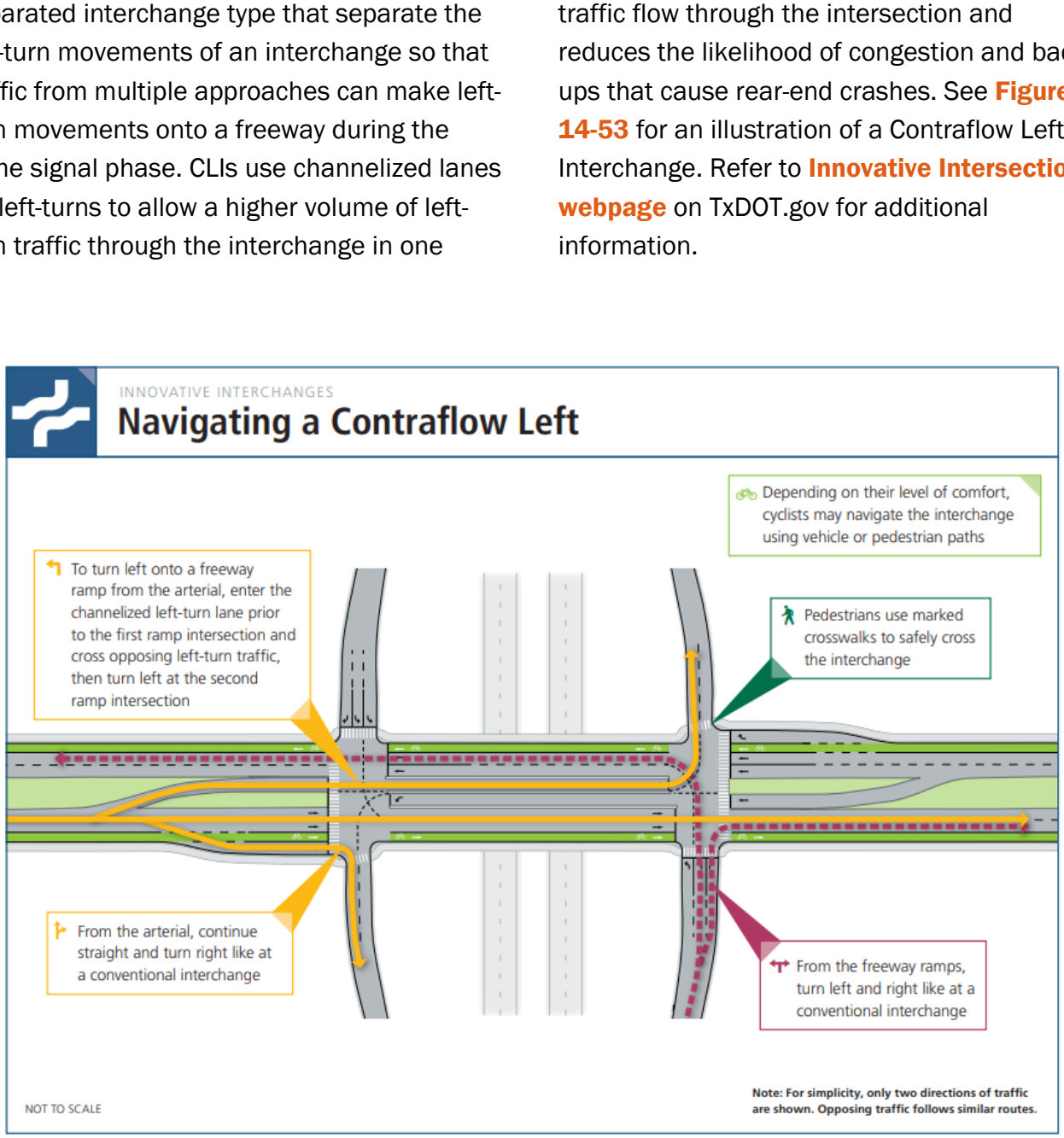


Figure 14-53: Example Contraflow Left Interchange

Source: VDOT (used by permission)

14.12 Single-Point Urban Interchange

Single-Point Urban Interchanges (SPUI), also known as single-point diamond interchanges (SPDI), reduce conflict points and traffic signal phases at an intersection by allowing opposing left-turn movements to proceed simultaneously without directly crossing paths of travel. This increases the capacity of the traffic signal by allowing more vehicles to clear the intersection in one signal cycle. SPUIs improve travel time and safety by having the turning movements of the major road ramps and the minor road executed at one central intersection rather than one on either side of the interchange. They are efficient where balanced left-turn volumes are present at an interchange. A SPUI can be advantageous along a corridor of closely-spaced signalized intersections.

See **Figure 14-54** and **Figure 14-55** below for an example SPUI and the corresponding left-turn and U-turn movements accommodated by a SPUI. Refer to the **Innovative Intersections webpage** on TxDOT.gov for additional information.

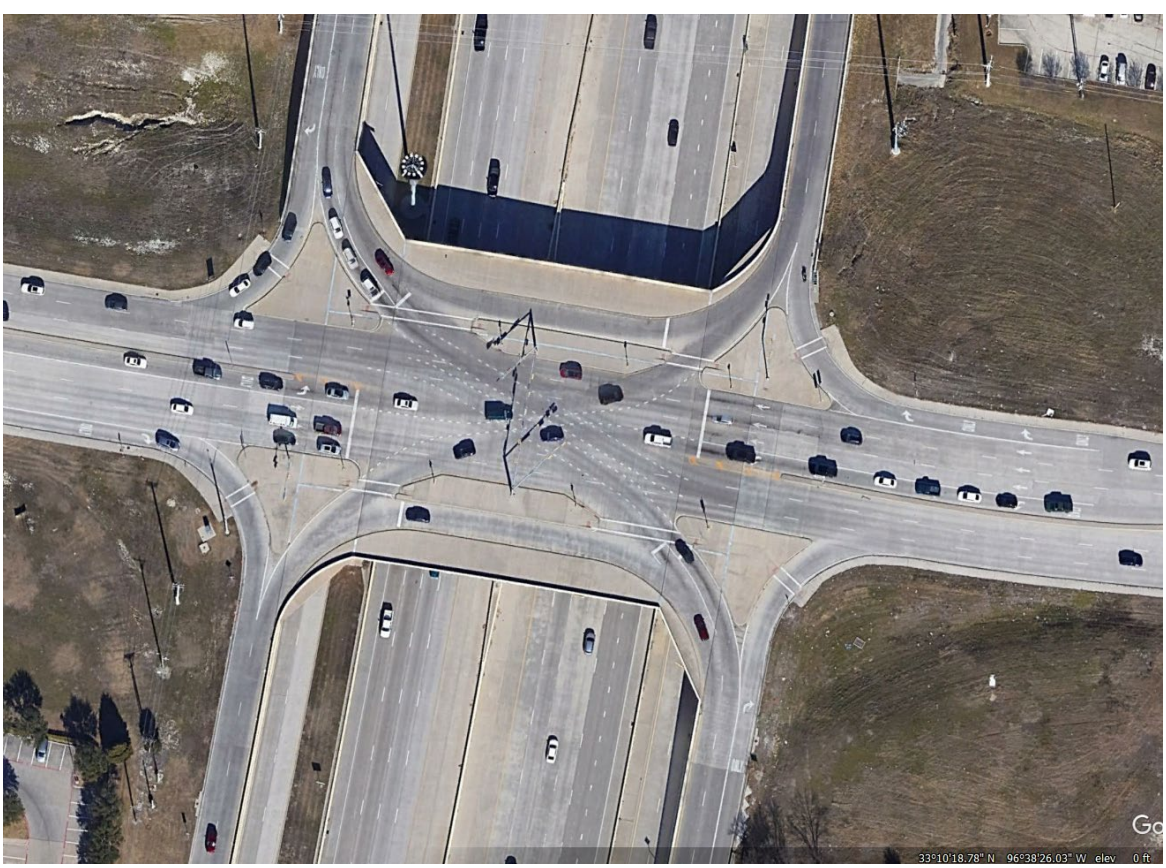


Figure 14-54: Example SPUI in McKinney, TX

Source: Google Earth



The second movement involves opposing left-turn lanes from the frontage road and/or exit traveling through the traffic signal onto the cross street.



The last movement is for the second set of opposing left-turn lanes traveling from the cross street onto the frontage road. Through-traffic on the frontage road is handled via intersection bypass lanes in both directions.

Figure 14-55: Left-Turns and U-Turns at a SPUI

Source: TxDOT Conceptual Concept

14.13 Roundabout Interchange

Roundabouts are routinely implemented at interchange ramp terminals as a pair of roundabouts, or a single-circle roundabout, and can be installed at interchanges with frontage roads as a means of collecting and distributing traffic within this type of context. Within urban areas, the existing or forecasted traffic volume at an interchange may likely exceed the capacity of a two-lane roundabout. Special consideration must be given to high-volume roundabout interchange studies and schematics to confirm the feasibility and reasonability of a roundabout interchange.

A major advantage roundabout interchanges provide is a reduced bridge width over the freeway due to the omission of auxiliary turn lanes that are often required at signalized interchanges. **Figure 14-56** shows the relative width of bridge needed to accommodate one lane in each direction across the interchange.

The reduction in bridge width can lead to environmental, construction duration, and cost benefits. Another advantage is the prevention of wrong-way maneuvers. As drivers circulate within the roundabout the entry lanes are angled in such a way that discourages drivers from making a right-turn and exiting the roundabout in the wrong direction.

When roundabout interchanges are installed where capacity is not exceeded during the peak period, the resulting level of service will typically far exceed that of other methods of interchange control due to off-peak periods operating at or near free-flow conditions. Minimal delay is achieved for 22+ hours of the day in this scenario.

Refer to **NCHRP Report 1043** and RDM **Section 14.4** for additional roundabout information.



Figure 14-56: Example Roundabout Interchange in Allen, TX

14.14 Partial Cloverleaf Interchange

A partial cloverleaf interchange is a standard cloverleaf interchange without ramps in all four quadrants. Partial cloverleaf interchanges can be used when site constraints limit the number of loops at the interchange. At-grade railroads, limited ROW availability, and water features are a few of the typical constraints that result in a partial cloverleaf interchange.

Advantages of partial cloverleaf interchanges consist of:

- Accommodating a heavy frequency movement from one freeway to another freeway with free-flow operations;
- The absence of third-party control devices to facilitate the transfer of vehicles from one roadway to another; and
- Driver familiarity due to the use of cloverleaf interchanges throughout the U.S.

One disadvantage of a partial, or full cloverleaf interchange, is the vast footprint required to incorporate the looping connections to and from the overlapping roadways. In lieu of cloverleaf interchange design, direct connectors and/or conventional diamond interchanges are more common in Texas in urban areas due to the benefits of a slightly reduced ROW footprint need and the accommodation of continuous frontage road. Rural areas see more cloverleaf installations where ROW may be more available. See **Figure 14-57** for an example of a partial cloverleaf interchange directional routing as compared to a full cloverleaf interchange routing.

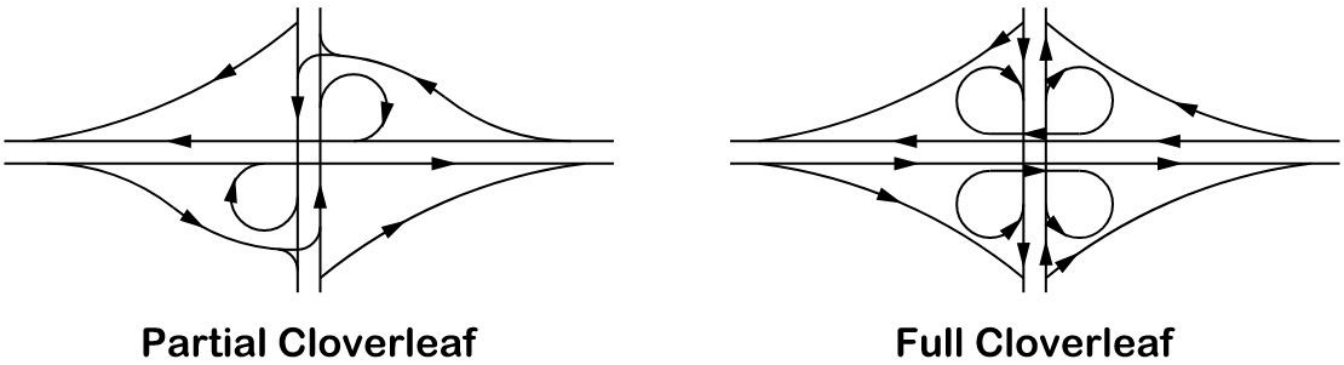


Figure 14-57: Partial and Full Cloverleaf

Source: FHWA Office of Safety

Chapter 15
Grade Separations and Interchanges

15.1 Overview

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.

- **Grade separation** - refers to the crossing of two roadways by a physical separation so that neither roadway interferes with the other.
- **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.

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.

For information on the design of Diverging Diamond Interchanges, see **Chapter 14**.

This chapter discusses the design of grade separations and interchanges and includes the following sections and sub-sections:

15.1 Overview

15.2 Warrants for Interchanges and Grade Separations

15.3 Types of Interchanges
- 15.3.1 Three-Leg Interchanges; and
- 15.3.2 Four-Leg Interchanges.

15.4 Applications on Various Type of Facilities
- 15.4.1 Urban Street Grade Separations;
- 15.4.2 Multi-Lane Rural Highway Grade Separations; and
- 15.4.3 Freeway Grade Separations.

15.5 Safety
- 15.5.1 Major Intersections with High Traffic Volumes;
- 15.5.2 Railroad Crossings; and
- 15.5.3 Benefits.

15.6 General Design Considerations

- 15.6.1 Interchange Spacing;
- 15.6.2 Lane Balance;
- 15.6.3 Auxiliary Lanes;
- 15.6.4 Weaving Sections; and
- 15.6.5 Collector-Distributor Roads.

15.7 Ramps & Direct Connectors

- 15.7.1 Type of Ramps;
- 15.7.2 General Ramp Design Considerations;
- 15.7.3 Ramp Geometrics;
- 15.7.4 Cross Section and Cross Slopes;
- 15.7.5 Lane and Shoulder Widths;
- 15.7.6 Sight Distance;
- 15.7.7 Ramp Terminal Design;
- 15.7.8 Ramp Spacing; and
- 15.7.9 Metered Ramps.

15.8 Frontage Road Turnarounds and Intersections

- 15.8.1 Guidelines for Designing Auxiliary Lanes (AL) on Frontage Road Approaches

15.2 Warrants for Interchanges and Grade Separations

Interchanges are integral features of freeways and are an adaptable solution to improve intersection conditions by improving safety, operational, and traffic congestion. Grade separations without ramps are also integral features on freeways. They can also provide significant system benefits to other facilities.

Since various factors come into play at each location, specific warrants for justifying an interchange cannot be conclusively stated. There are several conditions that need to be considered when deciding whether to use an interchange as a transportation solution. These include:

- **Design designation**—if the goal is to develop a highway with full access control between selected locations.
- **Reduction of bottlenecks or spot congestion**—if insufficient capacity at the intersection of heavily traveled routes results in intolerable congestion on one or all approaches.
- **Reduction of crash frequency and severity**—if some intersections have a very high rate of serious crashes, and inexpensive methods of reducing crashes are likely to be ineffective or impractical.
- **Site topography—**if grade-separation designs are the only type of intersection that can be constructed economically.
- **Road-user benefits**—if the road-user costs from delays at congested intersections are very high. Road-user costs include:
    - Fuel and oil usage;
    - Wear on tires;
    - Delay to motorists; and
    - Crashes that result from speed changes, stops, and waiting.
- **Traffic volume warrant**—if a traffic volume warrant may be the most tangible interchange warrant. Although a specific traffic volume at an intersection cannot be used as the only measure to determine the need for an interchange, it is an important variable. This is especially the case when combined with the traffic pattern and the effect of traffic behavior.

Not all warrants for grade separations are included in the warrants for interchanges. Additional warrants for grade separations include ones that will:

- Serve local roads or streets that cannot end outside the right-of-way (ROW) limits of freeways;
- Provide access to areas not served by frontage roads or other means of access;
- Eliminate a railroad–highway grade crossing;
- Serve unusual concentrations of pedestrian traffic (e.g., a city park developed on both sides of a major roadway);
- Serve bikeways and routine pedestrian crossings;

- Provide access to mass transit stations within the borders of a major roadway; and
- Provide free-flow operation of certain ramp designs and serve as part of an interchange.

Refer to Chapter 10 of **AASHTO's A Policy on Geometric Design of Highways and Streets** for detailed discussion on things to consider for each interchange and grade separation warrant.

15.3 Types of Interchanges

Interchanges are generally classified according to the number of approach roadways or intersection legs:

- Three-Leg: Tee or Trumpet interchange;
- Four-Leg: Diamond or Cloverleaf interchange;
- Multi-Leg interchanges; and
- Directional interchanges: 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.

15.3.1 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.

15.3.1.1 Trumpet

The most widely used three-leg interchange is the trumpet, as shown in **Figure 15-1**. 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.

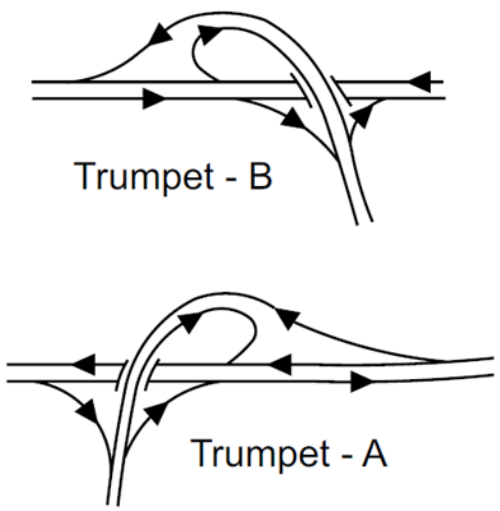


Figure 15-1: Trumpet Three Leg Interchange

15.3.1.2 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 15-2**.

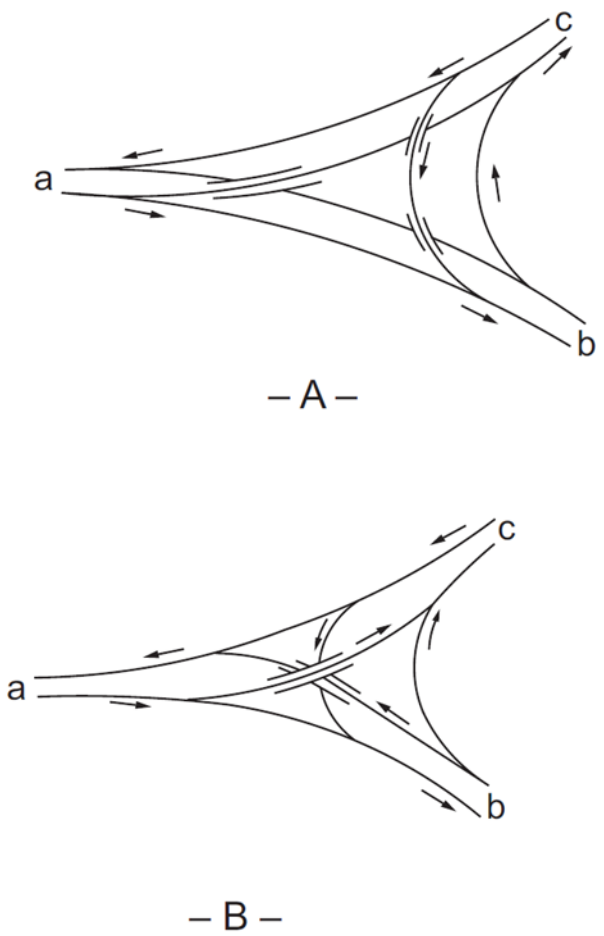


Figure 15-2: Directional Three Leg Interchange

15.3.2 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, ROW requirements, and system considerations. The three primary types of four-leg interchanges are Diamond, Cloverleaf, and Directional.

15.3.2.1 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 15-3.** Provide proper traffic signal timing if the 300-ft cannot be met.

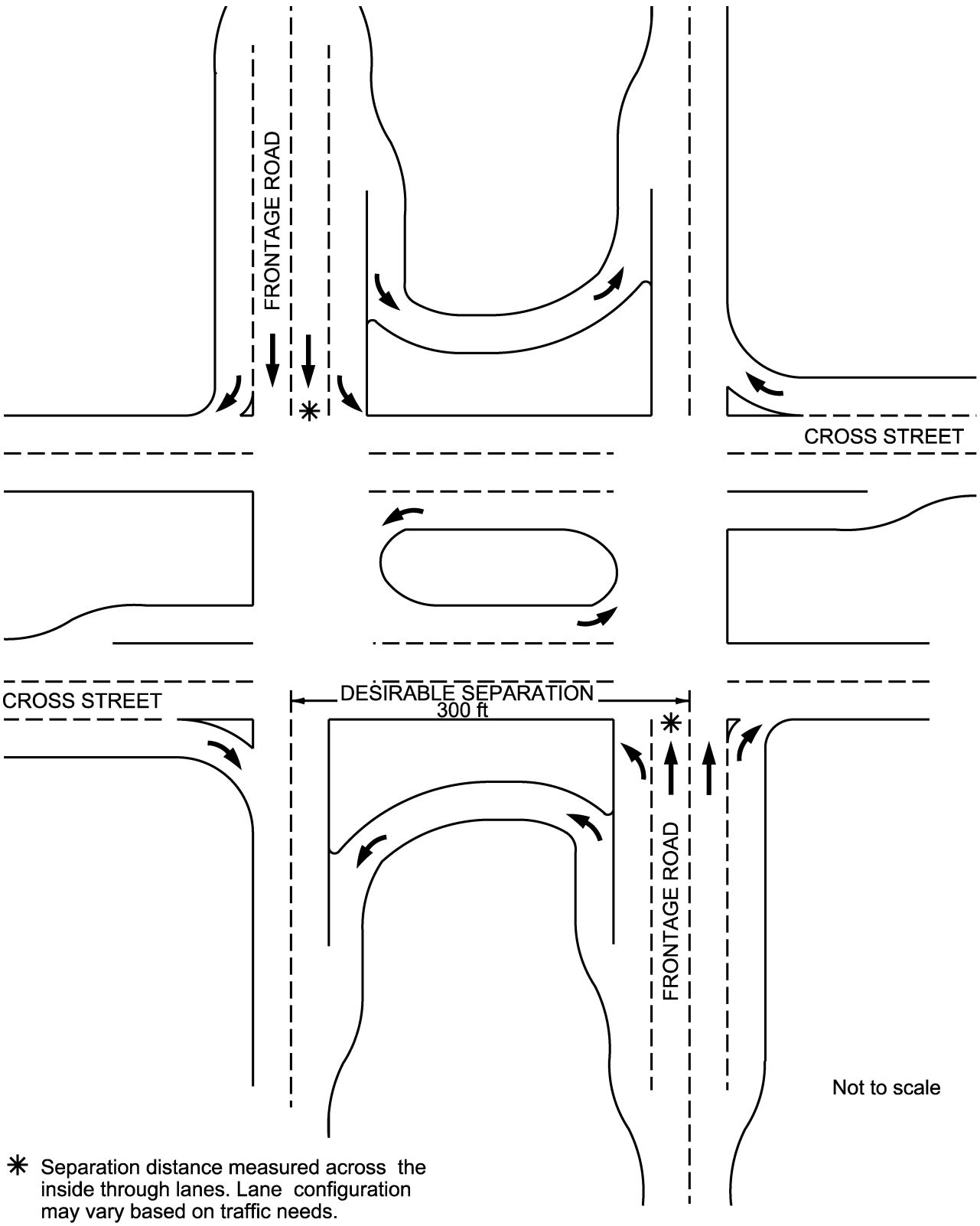


Figure 15-3: Separation of Frontage Roads in Diamond Interchanges

The diamond interchange may have several configurations, as discussed in the following paragraphs and shown in the figures below:

- **Conventional diamond without frontage roads -** The conventional diamond (**Figure 15-4**) 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.

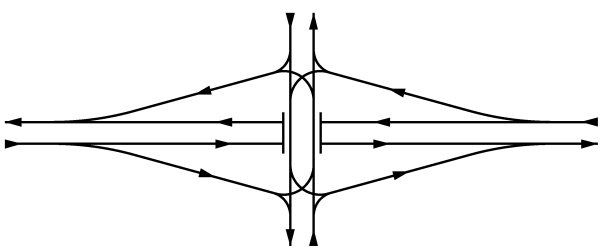


Figure 15-4: Conventional Diamond Interchange without Frontage Roads

- **Conventional diamond with frontage roads -** The conventional diamond with frontage roads (**Figure 15-5**) 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. Queues form on the ramp or frontage due to vehicles going through the intersection to gain access to the property located along the frontage road.

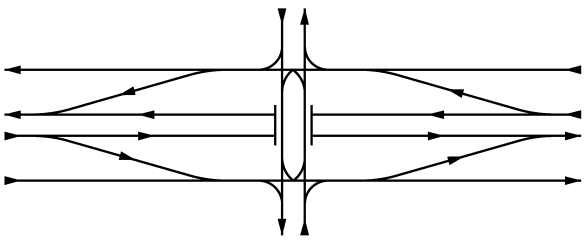


Figure 15-5: Conventional Diamond Interchange with Frontage Roads

- **Spread diamond** - The spread diamond (**Figure 15-6**) 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 ROW required may limit its usage.

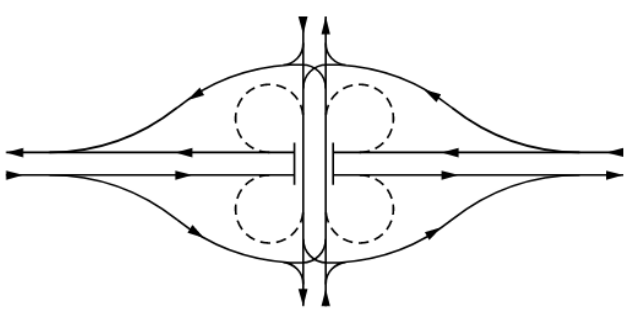


Figure 15-6: Spread Diamond Interchange

- **X- pattern** - The X-pattern interchange (**Figure 15-7**) 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.

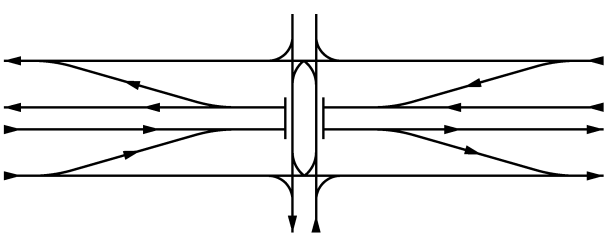


Figure 15-7: X-pattern Interchange

- **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". Both variations are shown in **Figure 15-8**.

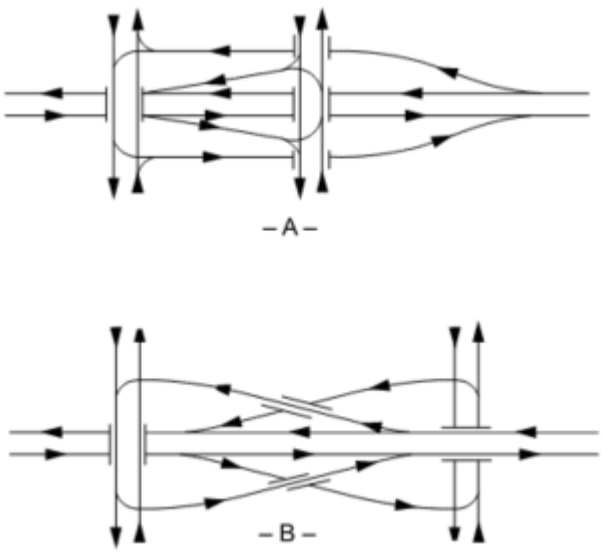


Figure 15-8: Stacked Diamond Interchange

Source: AASHTO A Policy on Geometric Design of Highways and Streets

- **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 15-9**). 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.

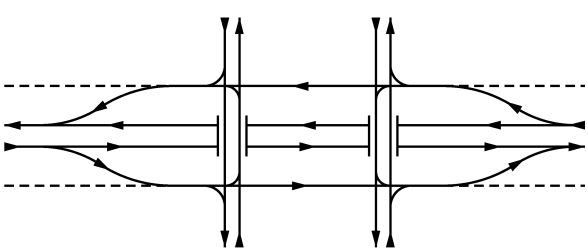


Figure 15-9: Split Diamond Interchange

- **Split diamond with one-way cross streets and frontage roads** - This layout is simplistic in layout and operation of both the crossroad and the at-grade terminals (**Figure 15-10**). Traffic leaving the freeway is afforded easy access to return to the freeway and continue the journey in the same direction.

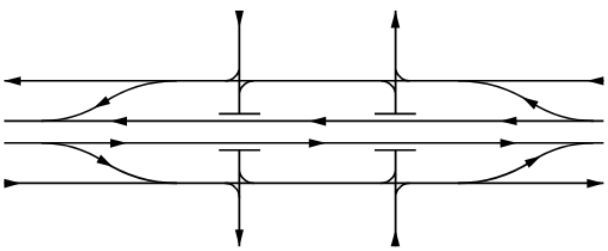


Figure 15-10: Split Diamond with One-way Cross Streets and Frontage Roads

- **Diamond interchange with frontage roads and separate turnaround provisions** - These are highly desirable if the cross street has heavy traffic volumes and there is considerable demand for the turnaround movement (**Figure 15-11**). The turnaround roadways are adjacent to the cross street with additional width provided beneath the structure or, if the cross street overpasses the freeway, on top of the structure. See **Section 15.8** for more information on frontage road turnarounds and intersections.

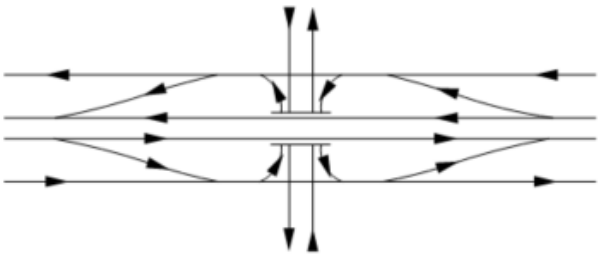


Figure 15-11: Diamond Interchange with Frontage Roads and Turnarounds

- **Three level diamond** - In urban areas, where the cross street carries a high volume of traffic, the three-level diamond interchange, illustrated in **Figure 15-12,** 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. Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for design details.

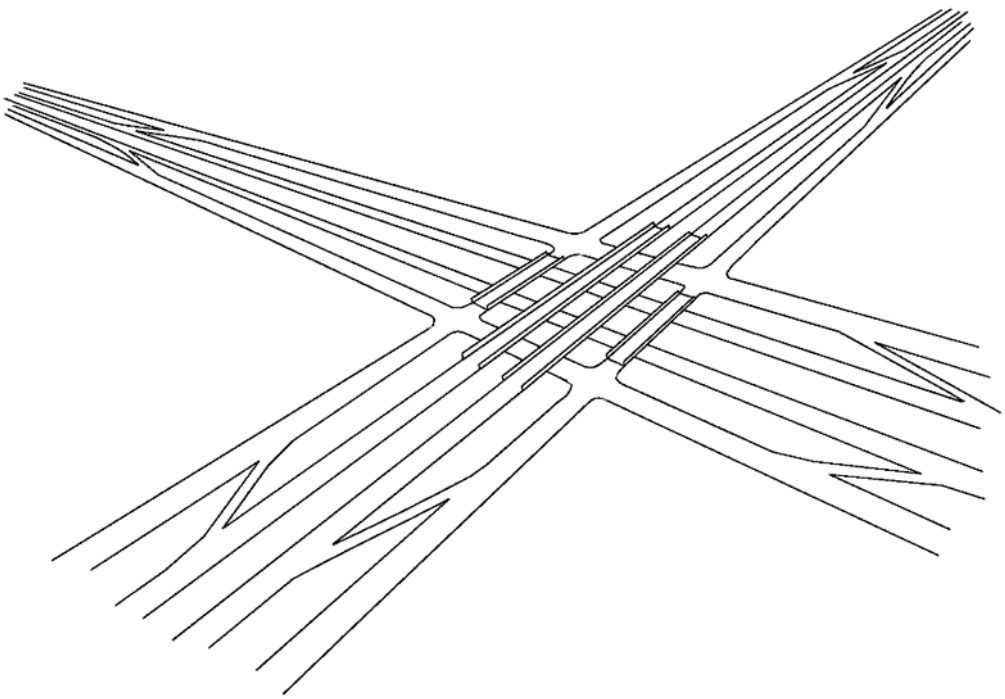


Figure 15-12: Three Level Diamond Interchange

- **Single point diamond** - A special type of freeway-to-arterial interchange has received attention during recent years and is worthy of discussion. Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for "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 15-13**. 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.

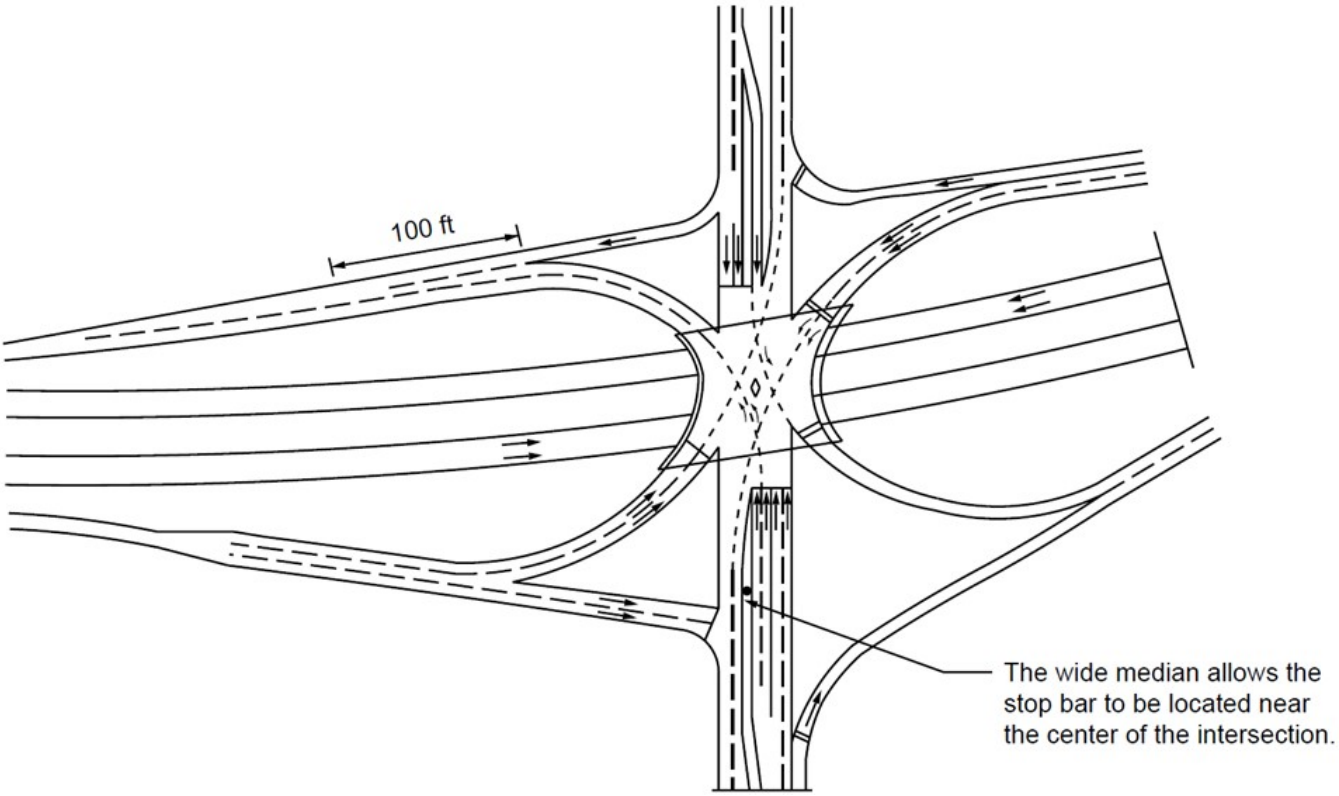


Figure 15-13: Split Point Diamond Interchange

- **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 15-14**. 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.

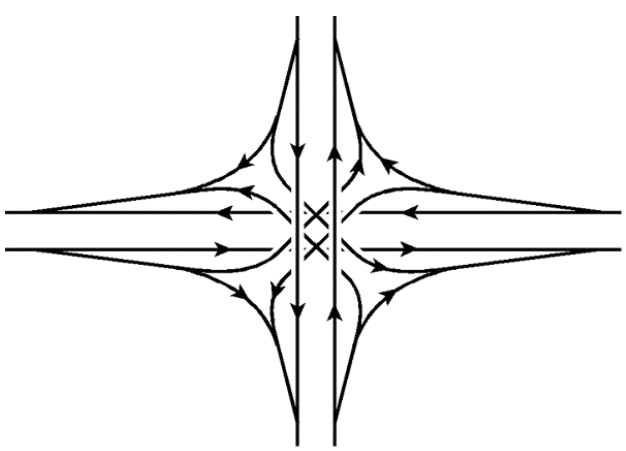


Figure 15-14: Three Level Stacked Diamond Interchange

As indicated in **Figure 15-15**, 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.

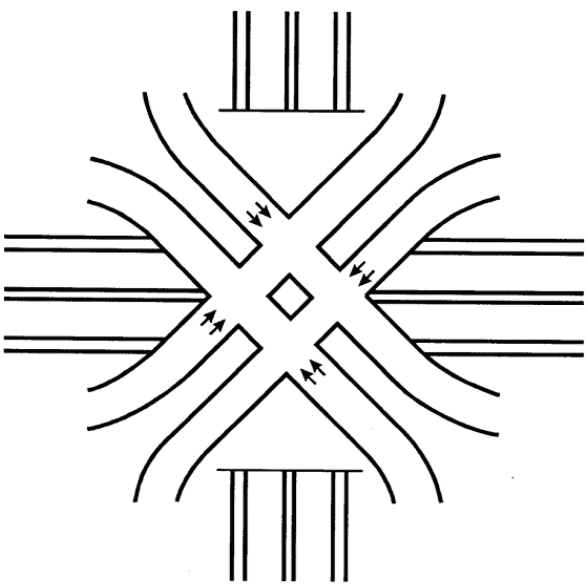


Figure 15-15: Three Level Stacked Diamond At-Grade Interchange

15.3.2.2 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 ROW 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, see **Section 8.1.19.**

- **Full cloverleaf** - The four-quadrant, full cloverleaf, illustrated in **Figure 15-16**, eliminates all left-turn conflicts through construction of a two-level interchange.

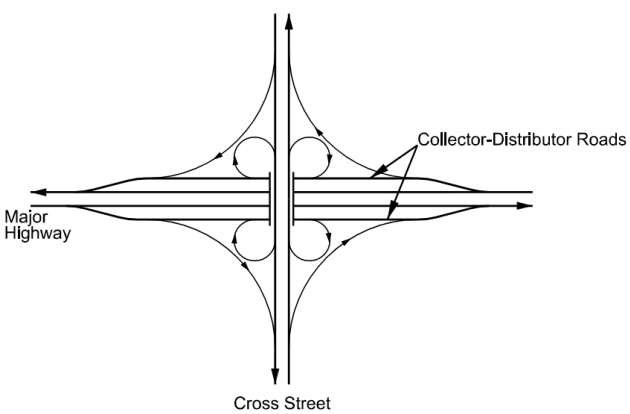


Figure 15-16: Full Cloverleaf Interchange

- **Partial cloverleaf** - A cloverleaf without ramps in all four quadrants, illustrated in **Figure 15-17**, 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 referred to in **AASHTO's A Policy on Geometric Design of Highways and Streets.**

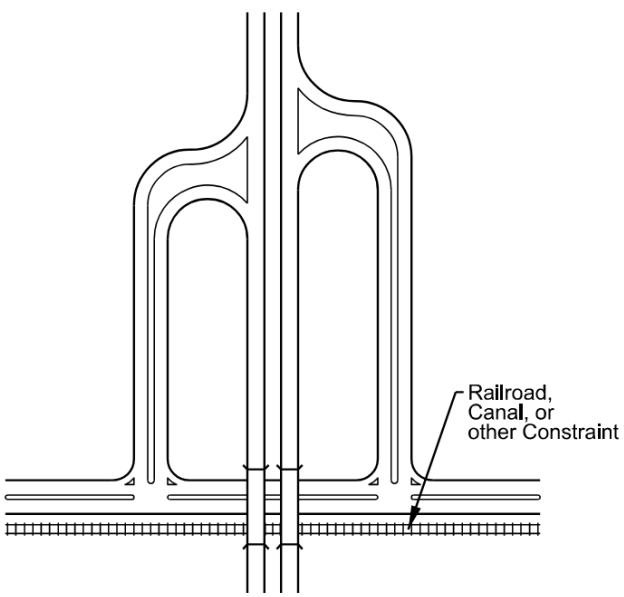


Figure 15-17: Partial Cloverleaf Interchange

15.3.2.3 Directional Interchanges

Interchanges that use direct or semi-direct connections for one or more left-turn movements are called "directional" interchanges (**Figure 15-18** and **Figure 15-19**).

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.

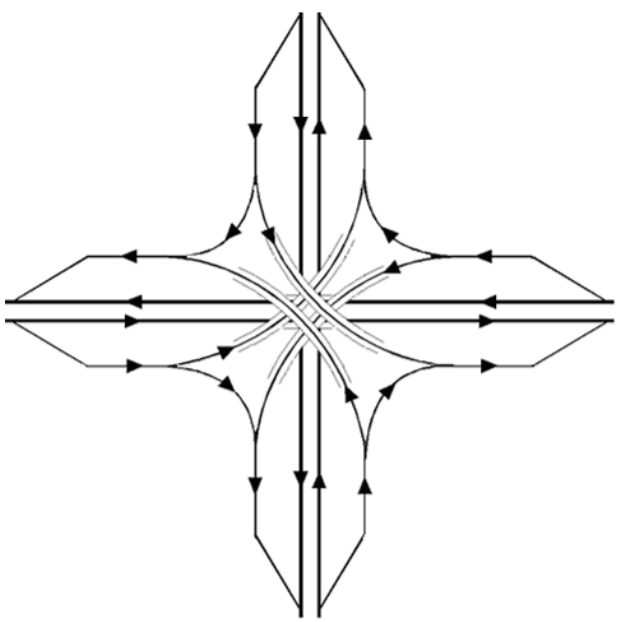


Figure 15-18: Four Level Fully Directional Interchange without Frontage Roads

- **Four level without frontage roads** - The four-level directional interchange as depicted in **Figure 15-18** 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 15-19** depicts a five-level interchange with frontage roads.

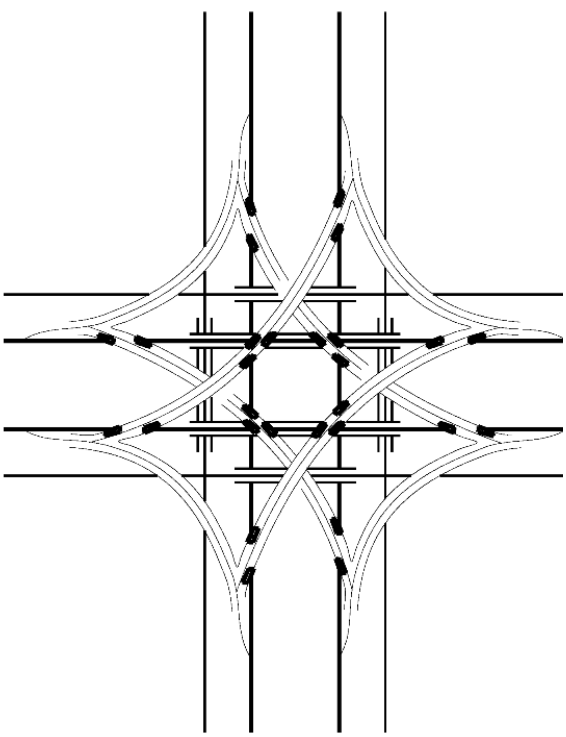


Figure 15-19: Five Level Fully Directional Interchange with Frontage Roads

- **Turbine interchange** - This interchange is an alternative four-way directional interchange. It requires fewer levels (usually two or three) while retaining direction ramps throughout. It features right-exit, left-turning ramps that sweep around the center of the interchange in a clockwise spiral. **Figure 15-20** depicts a typical turbine interchange.

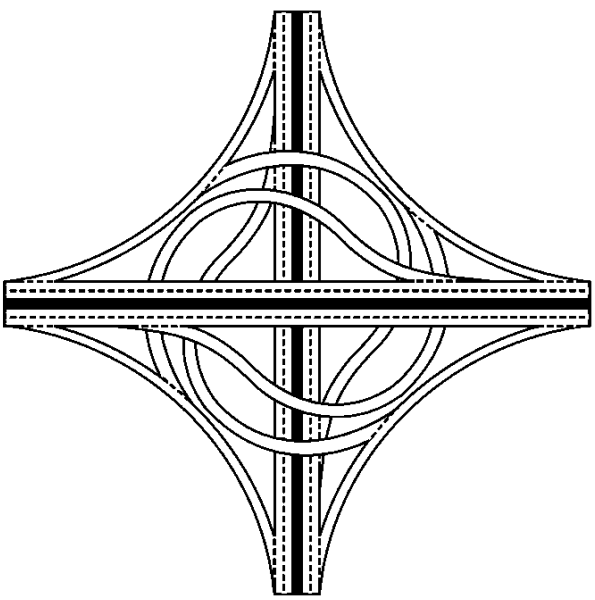


Figure 15-20: Turbine Interchange

15.4 Applications on Various Type of Facilities

15.4.1 Urban Street Grade Separations

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. Except for 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 Chapter 10 of the **TSAP** and **FHWA Intersection Control Evaluation (ICE)** for additional information on optimal geometric configurations and traffic control solutions for interchanges.

Where applicable, 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 ROW availability permits. It is not desirable to intermix facility types by providing intermittent sections of fully controlled and non-controlled access facilities.

15.4.2 Multi-Lane Rural Highway Grade Separations

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

15.4.3 Freeway Grade Separations

The remaining sections of this chapter discuss grade separations and interchanges as they pertain to Freeways since this is where these types of solutions are most common and useful.

For additional discussion on grade separations and interchanges, refer to Chapter 10 of **AASHTO's A Policy on Geometric Design of Highways and Streets.**

15.5 Safety

At-grade intersections have the potential for crashes resulting from vehicle-vehicle and vehicle-pedestrian conflicts, in part due to conflicting crossing and turning movements that happen in a limited area. Grade separating these conflict points, using overpasses or underpasses, allow traffic to flow freely, in turn, making conditions safer for vehicles and pedestrians as well as trains.

Three roadway improvement objectives are achieved using grade-separated intersections:

- Increased capacity and constant flow;
- Increased safety; and
- Reduced vehicle–train conflict and delay.

Overpasses increase the capacity of a roadway by allowing constant flow in all directions. The intersection approaches bypass each other thereby avoiding direct contact. Fewer signals are needed to direct traffic, thus decreasing the backup caused by signals.

Grade separation increases roadway safety by reducing vehicle–vehicle and vehicle–pedestrian conflicts. The crossing traffic is removed from the intersection, which lowers the chance of collisions. Pedestrians are better protected from cars since they only have to cross one line of traffic. Also, more refuge points can be provided at multiple locations.

Removing intersections that serve both trains and vehicles substantially increases speeds and road capacity. Street traffic moves freely over or under the railroad tracks, eliminating wait times for a passing train. Most importantly, train–vehicle collisions are eliminated since the intersection no longer puts traffic in front of trains.

15.5.1 Major Intersections with High Traffic Volumes

Intersections are a large cause of congestion on major streets. The signal time given to each direction decreases a road's capacity, increasing the possibility of congestion. The stop-and-go condition reduces safety and increases travel time for all drivers. Elevating one of the streets reduces the conflict caused by intersecting roadways. The reduced conflict increases the road capacity.

15.5.2 Railroad Crossings

Railroad crossings pose a threat to the safety of street and rail traffic. Same-level crossings, especially near intersections, create many conflict points for cars, pedestrians, and trains. These crossings force both cars and trains to slow down, increasing travel time and congestion for cars and decreasing overall efficiency for the rail network.

15.5.3 Benefits

Grade-separated intersections increase capacity by eliminating the delay caused by a previous intersection or railroad. Traffic moves freely, and signals may only be necessary for accessing the exit and entrance ramps of the interchange.

Elevating one portion of a street or rail crossing improves safety by eliminating vehicle, pedestrian, and train conflicts. The chance of collisions is decreased because:

- Crossing traffic is minimized;
- Trains are separated from the roadway; and
- Pedestrian conflict points are reduced.

Refer to the **HSM** reports for the benefits of changing to a grade-separated interchange. Converting a four-way stop to a grade separated intersection reduces injury crashes by 57 percent. Converting a signalized intersection reduces injury crashes by 28 percent.

15.6 General Design Considerations

15.6.1 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 recommended 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 the centerline of cross streets as shown in **Figure 15-21**.

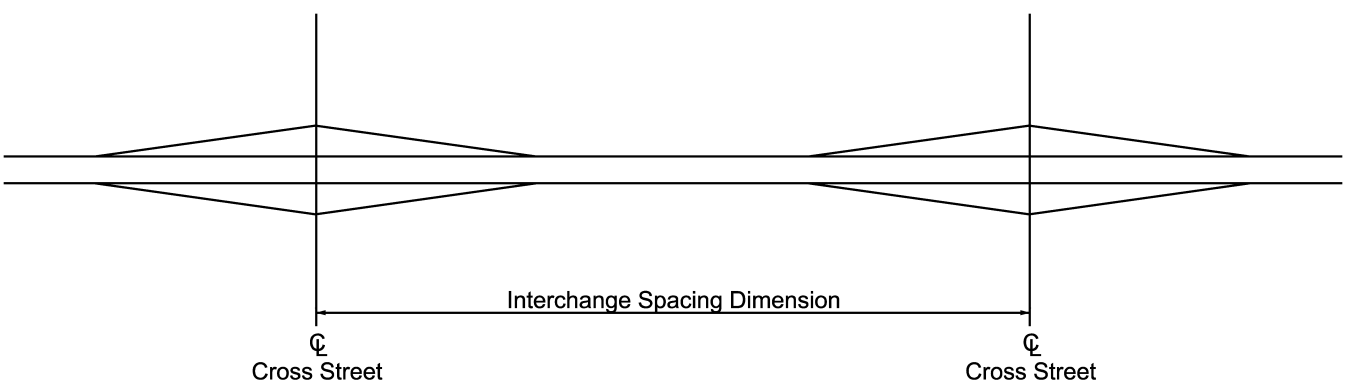


Figure 15-21: Interchange Spacing Between Cross Streets

15.6.2 Lane Balance

See **Section 8.1.6** for a detailed discussion of lane balance on Freeways and their application to grade separations and interchanges.

15.6.3 Auxiliary Lanes

A continuous auxiliary lane should be constructed between the entrance and exit terminals of interchanges where 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.

An auxiliary lane may be introduced as a single exclusive lane or in conjunction with a 2-lane entrance. Refer to **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 through the procedures can be referred to in the **HCM** which is applicable to all freeway contexts.

See **Chapter 8, Section 8.1.8** for additional information regarding auxiliary lanes.

15.6.4 Weaving Sections

Weaving sections occur when entrance ramps are closely followed by exit ramps and vehicle paths cross each other. Such areas present special design problems due to the concentrated lane changing maneuvers of merging and diverging traffic. The development of the design involves the following factors:

- Desired mobility standard;
- Length;
- Number of lanes;
- Traffic volumes;
- Weaving and non-weaving vehicles; and
- Average speed.

For design guidance refer to Design Controls and Criteria, Chapter 2 of **AASHTO's A Policy on Geometric Design of Highways and Streets** and from Chapter 24 of the **HCM**.

See **Section 15.7.8** for additional information on weaving lengths and ramp spacing.

15.6.5 Collector-Distributor Roads

See **Section 8.1.19** for a detailed discussion of collector-distributor roads and their application to grade separations and interchanges.

Refer to Chapter 10 of AASHTO's A Policy on Geometric Design of Highways and Streets for discussion on other General Design Consideration topics.

15.7 Ramps & Direct Connectors

15.7.1 Types of Ramps

The term "ramp" includes all types, arrangements, and sizes of turning roadways that connect two or more legs at an interchange. The components of a ramp are a terminal at each leg and a connecting road, usually with some curvature, and on a grade. Generally, the horizontal and vertical alignment standards of ramps are below that of the intersecting highway, but in some cases it may be equal.

The basic types of ramps are:

- Diagonal;
- One quadrant;
- Loop and semidirect;
- Outer connection; and
- Directional.

For further information on these basic ramp types, refer to Chapter 10 of **AASHTO's A Policy on Geometric Design of Highways and Streets.**

15.7.2 General Ramp Design Considerations

15.7.2.1 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 ROW 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 15-1**. 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.** See **Figure 15-22** for the definition of gore area characteristics.

The design speed for a ramp should not be less than the design speed on the intersecting frontage roads. Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional guidance on the application of the ranges of ramp design speed shown in **Table 15-1**. Refer also to **NCHRP Report 313 Selecting Ramp Design Speeds, Vol 1 and 2 Guides** for more detailed analysis.

Table 15-1: 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 superelevation methodology see Table 4-3.
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.

15.7.3 Ramp Geometrics

15.7.3.1 Entrance Ramp Connections.

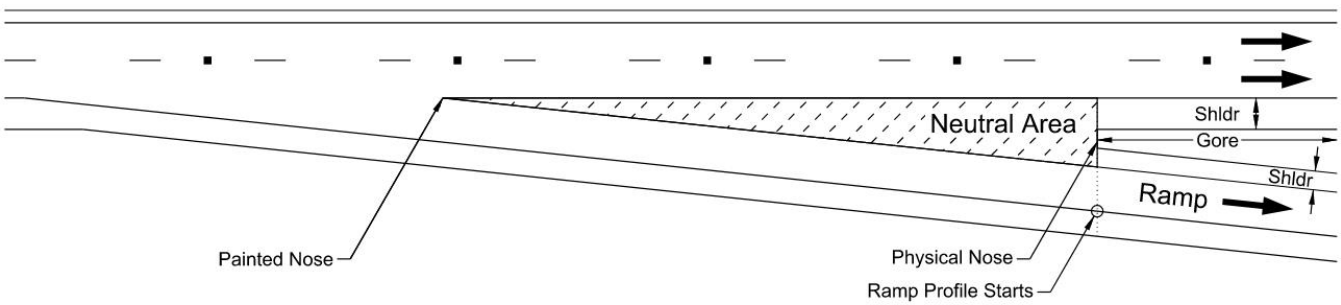
The horizontal geometry of entrance ramps will depend on the design speeds of the ramp, the connecting frontage road, and the connecting freeway. The departure from the frontage road should be a smooth transition appropriate for the design speed and speed classification (low speed/high speed) of the 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.

15.7.3.2 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.

15.7.3.3 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 15-22**.



* Width of physical nose should be based on local maintenance preferences.
* This figure is not intended to show striping. See latest FPM standard for signing and pavement markings.

Figure 15-22: 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.

15.7.3.4 Grades and Profiles.

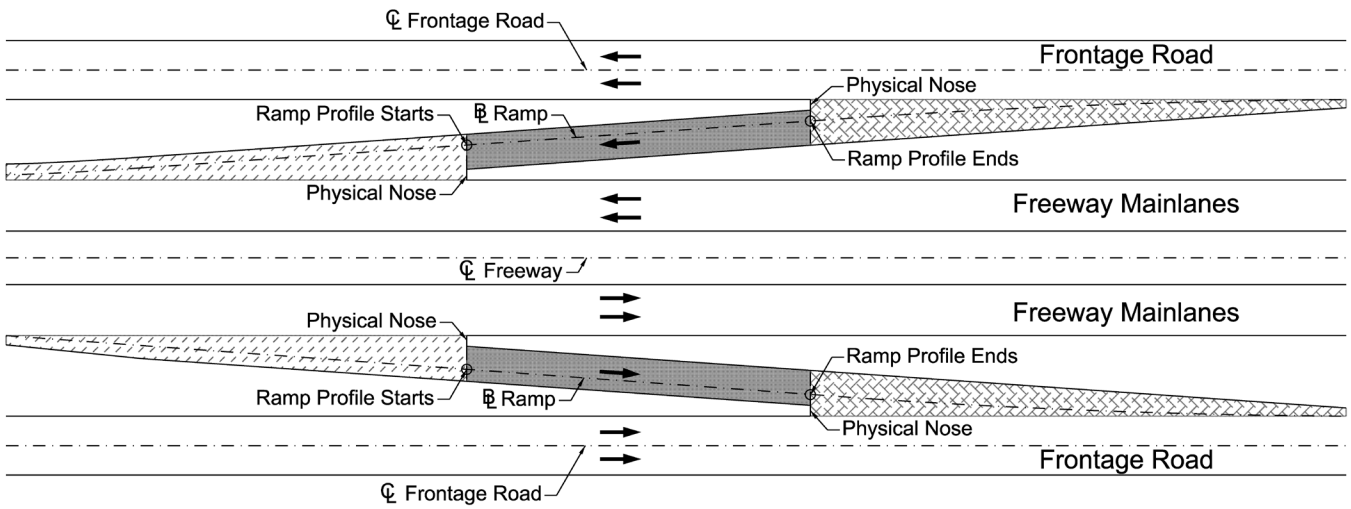
For design controls for crest and sag vertical curves on ramps and direct connectors, see **Figure 4-5** and **Figure 4-6**. Longer vertical curves with increased stopping sight distances should be provided wherever possible. See **Section 4.8.3** for guidance on grade breaks without a vertical curve.

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**. Maximum ramp grades are shown in **Table 15-2**. In general, these grades should be used infrequently and only when necessary. Engineering judgement should be exercised when there are certain geometric constraints or topographic conditions that permit the use of these maximum grades.

Table 15-2: 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 15-23** shows typical characteristics of ramp profile limits.



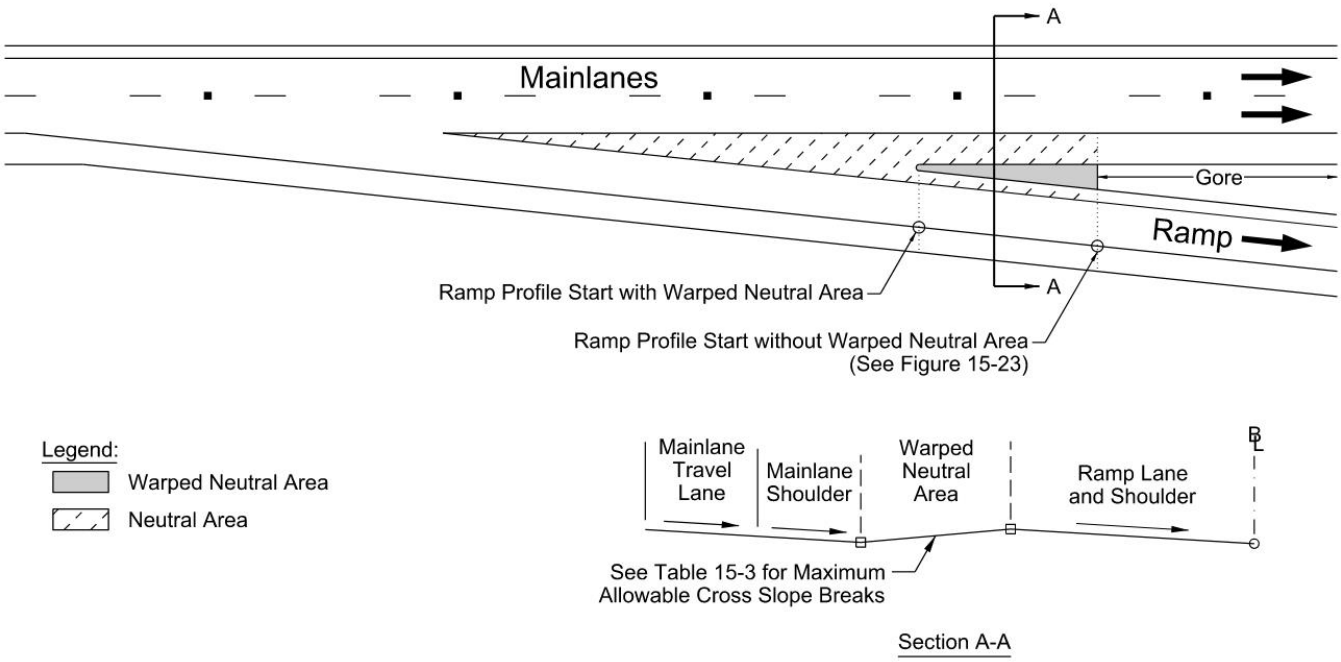
Legend:

- Independent Ramp Profile and Cross Slope
- Ramp Profile and Cross Slope Controlled by Freeway Mainlanes
- Ramp Profile and Cross Slope Controlled by Frontage Road

* This figure is not intended to show striping. See latest FPM standard for signing and pavement markings.

Figure 15-23: 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 15-24** for illustrations of a warped neutral area and **Table 15-3** for maximum algebraic differences in cross slope.



* This figure is not intended to show striping. See latest FPM standard for signing and pavement markings.

Figure 15-24: Warped Neutral Area

Table 15-3: 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

15.7.4 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 seen in **Chapter 4, Table 4-6** or **Table 4-7**.

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 15-4**;
- Lane balance; and

- Where traffic volumes require two lanes merging onto a freeway, adequate weaving distance must be provided.

15.7.5 Lane and Shoulder Widths.

Lane and shoulder widths for ramps and direct connectors are shown in **Table 15-4.**

Table 15-4: Ramp and Direct Connector Roadway and Shoulder Widths

	Inside Shoulder Width (ft.)	Outside Shoulder Width (ft.)	Lane Width (ft.)
1-Lane Direct Conn	2 or 4[1] Rdwy; 4[1] Str	8	14
2-Lane Direct Conn	2 or 4[1] Rdwy; 4[1] Str	8	12
Ramps (uncurbed)	2 or 4[1] Rdwy; 4[1] Str	6 to 8	14[2]
Ramps (curbed)	-	-	22[2]

Notes:

1. 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.
2. 12-ft lanes are permissible on ramps with multiple lanes.

15.7.6 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. See **Table 4-24** for 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. For **Decision Sight Distance**, see **Section 4.11.2,** for a desirable goal.

15.7.7 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; and**
- Exit Ramps to Frontage Roads.

15.7.7.1 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:

- The speed at which drivers merge with through traffic;
- The speed at which drivers enter the acceleration lane; and
- The manner of acceleration.

See **Table 4-18** for 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 4-19**. Point A (shown in **Figure 15-25**) 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.

15.7.7.2 Taper-Type Entrance Ramp to Freeway

A taper-type entrance shown in **Figure 15-25** 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.

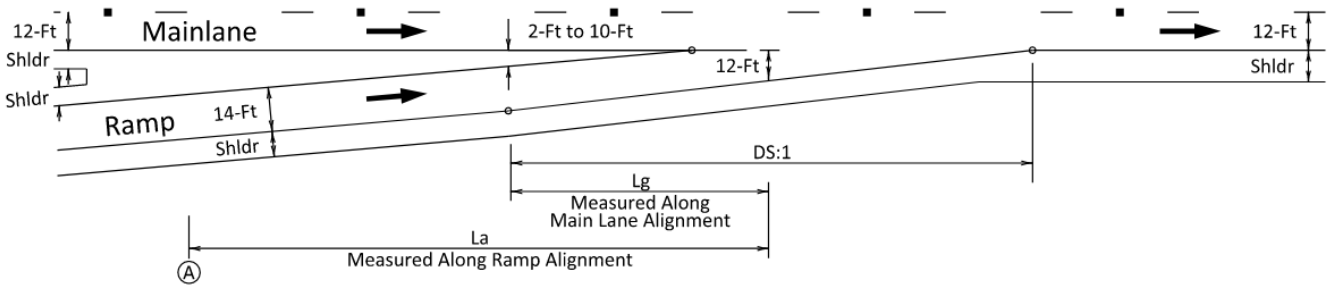
15.7.7.3 Parallel-Type Entrance Ramp to Freeway

The parallel-type entrance ramp to freeway shown in **Figure 15-25** 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.

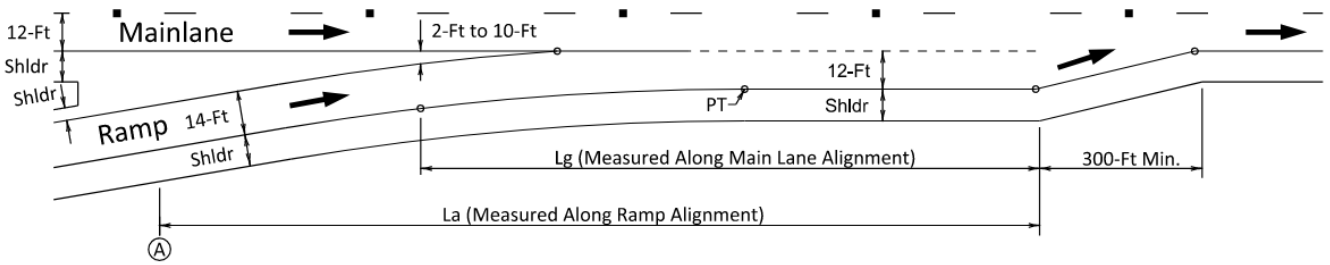
Desirably, a ramp curve with a baseline radius of 1,000-ft or more and length of at least 200-ft should be provided in advance of the added lane. If a radius less than 1,000-ft is used drivers tend to drive directly onto the freeway without using the acceleration lane. The taper at the downstream end 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.



Taper-Type Entrance Ramp



Parallel-Type Entrance Ramp

Notes:

1. La = Recommended length of acceleration as shown in Table 4-18 and adjusted in Table 4-19.
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.
6. For parallel-type entrance ramps, the inside edge line stripe is truncated to mainlane outside edge line stripe at the painted nose.

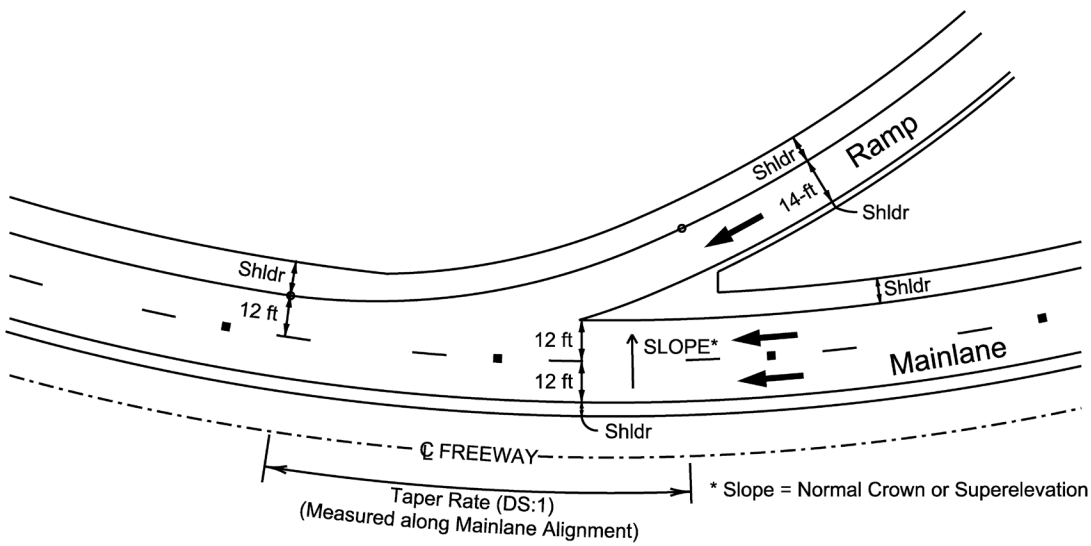
Figure 15-25: Single-Lane Entrance Ramp Terminals on a Tangent

15.7.7.4 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. Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for guidance on developing the alignment of a speed change lane on curves.

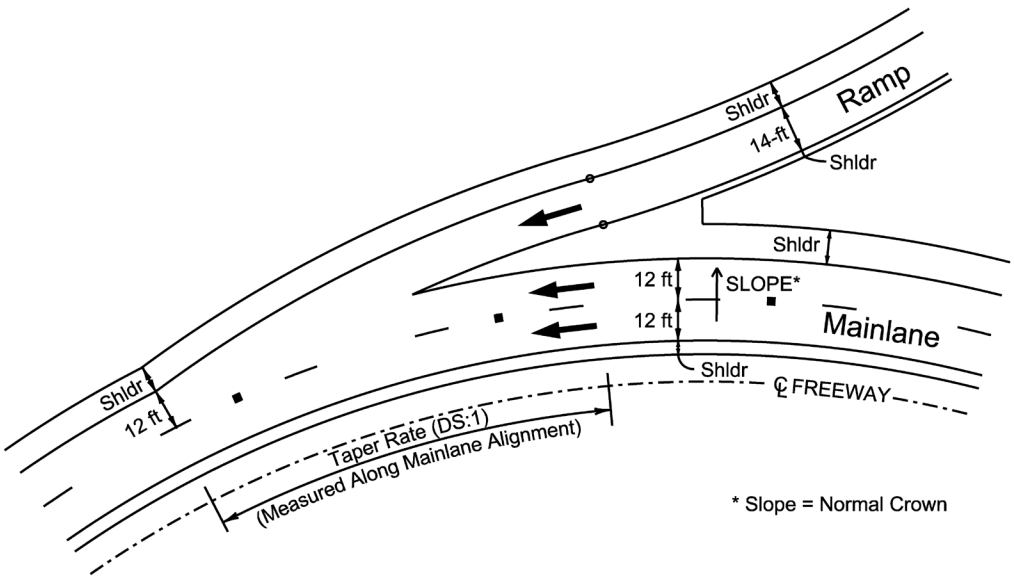
Figure 15-26 through **Figure 15-28** show different cross slope and curvature combinations of an entrance ramp connecting

to the mainlanes. In all scenarios, the rate of taper (DS:1) is measured against 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, a sufficient length of tangent or a larger compound curve must 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 throughout the curve to meet the desirable taper length.



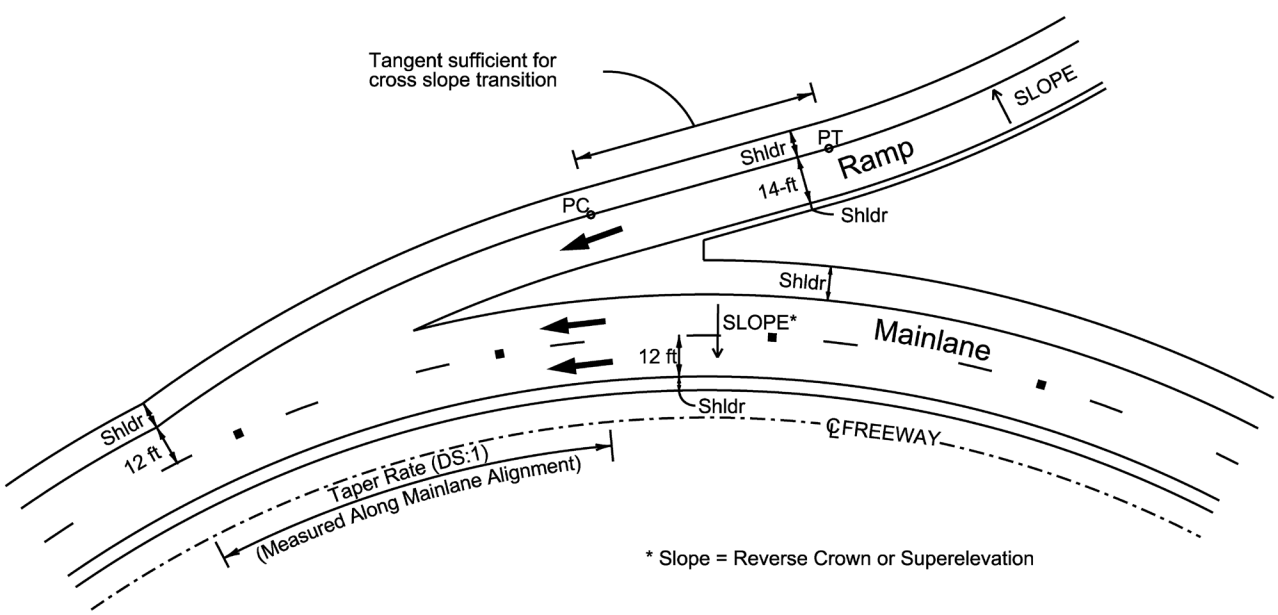
* This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 15-26: Entrance Ramp to a Freeway Curving in The Direction of The Ramp



* This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 15-27: Entrance Ramp to A Freeway Curving Away from The Direction Of The Ramp



* This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 15-28: Entrance Ramp to a Superelevated Freeway Curving Away from the Ramp

15.7.7.5 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 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.

15.7.7.6 Taper-Type Exit Ramp from Freeway

A taper-type exit ramp, as shown in **Figure 15-29** tangent example 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.

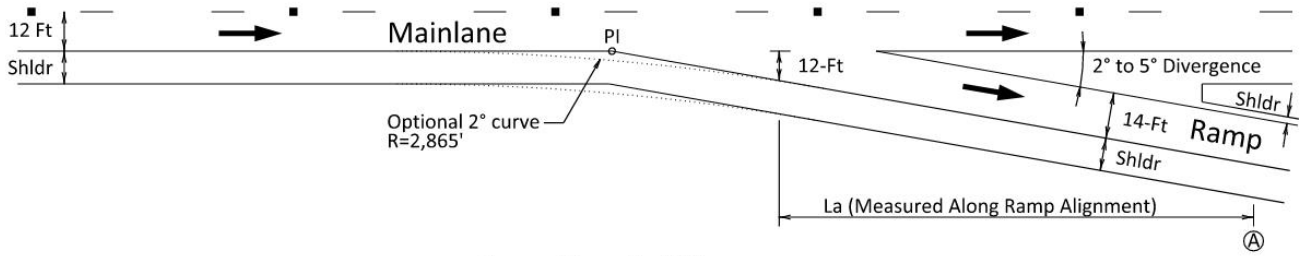
The taper-type exit may also be designed using an alignment as shown in

Figure 15-29 curvilinear example. **The divergence angle resulting from the designed alignment should be calculated and compared to the typical 2 to 5 degrees**.

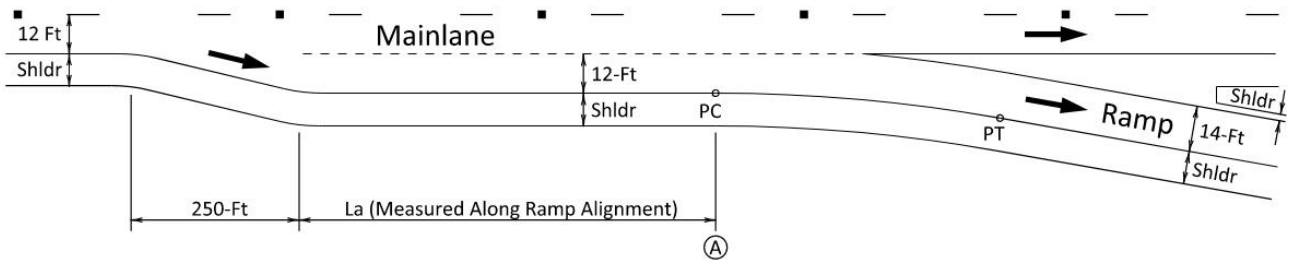
15.7.7.7 Parallel-Type Exit Ramp from Freeway

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 15-29**. 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 15-5**. For grade adjustments see **Table 4-19**. The values in **Table 15-5** 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.



Taper-Type Exit Ramp



Parallel-Type Exit Ramp

Notes:

1. La = as shown in Table 15-5 with adjustments in Table 4-19.
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 15-29: Typical Single-Lane Exit Ramp Terminals

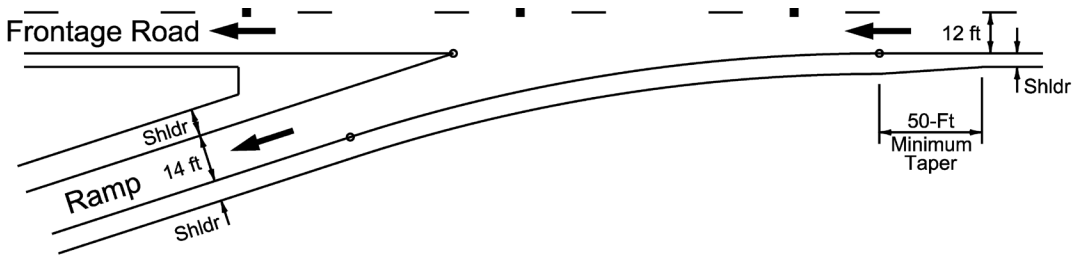
Table 15-5: Minimum Deceleration Lane Lengths for Exit Ramps with Flat Grades of Less Than 3 Percent

Deceleration Lane Length, L_a (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
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

15.7.7.8 Entrance Ramp from Frontage Road

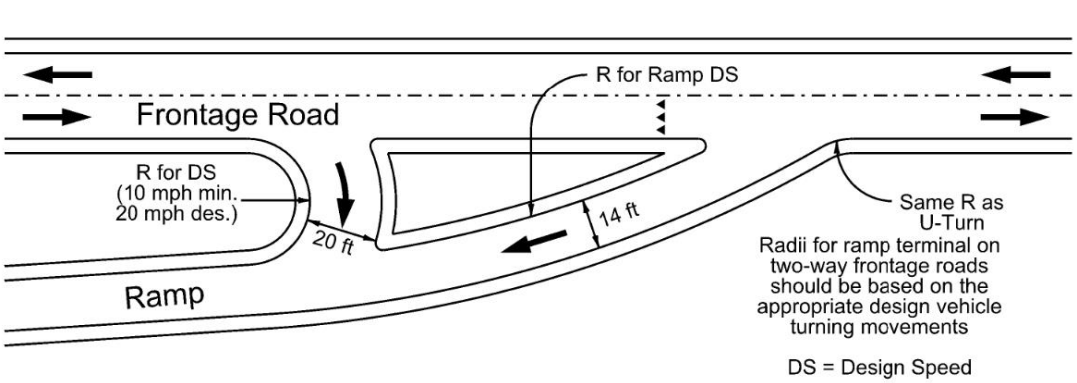
The geometry of ramps connecting from frontage roads will differ significantly depending on whether they are connecting to a one-way or two-way frontage road. Connection from one-way frontage roads is recommended, as this reduces the chance of wrong-way traffic on a ramp. The horizontal geometry of the ramp connecting from a frontage road should be equal to or more conservative than the frontage road dependent upon the context and speed, see **Table 4-3** for appropriate superelevation methodologies.

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 15-30** and **Figure 15-31.** For more geometry and access control information on entrance ramps from frontage roads, see **Figure 8-8.**



* This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 15-30: Taper-Type Entrance Ramp from a One-Way Frontage Road



* This figure is not intended to show striping or pavement marking details. See latest FPM standards for signing and pavement marking details.

Figure 15-31: Taper-Type Entrance Ramp from a Two-Way Frontage Road

15.7.7.9 Exit Ramps to Frontage Roads

Ramps should connect to the frontage road at the minimum distance (see **Table 8-2**). 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, see **Figure 8-7**.

15.7.8 Ramp Spacing

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 as illustrated in Figure 15-32.

The distance between successive ramps must be determined with the analysis procedures referred to in the HCM. A design waiver will be required if the minimum distance for the minimum Level of Service (LOS) as determined by an HCM analysis cannot be met. Additionally, a design waiver will be required if the minimum values in **Figure 15-32** cannot be met.

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, then speed-change lanes should be connected to provide an auxiliary lane.

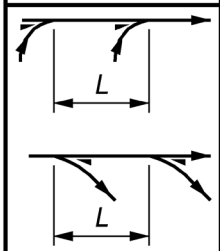
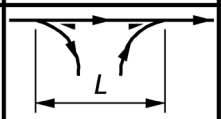
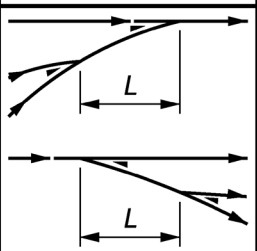
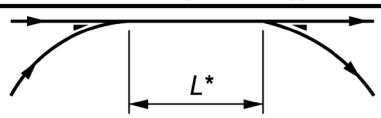
When the minimum needed distance cannot be achieved, consideration should be given to consolidating or braiding the ramps in lieu of requesting a design waiver.

Ramp configurations shown in **Figure 15-32** are relative to the facility being traveled on.

See section 15.8.1 for additional considerations on ramp spacing along frontage roads.

15.7.9 Metered Ramps

Where ramps are currently expected to accommodate metering, or expected to in the future, the geometric design features referred to in **AASHTO's A Policy on Geometric Design of Highways and Streets** 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. Refer to **AASHTO's A Policy on Geometric Design of Highways and Streets** for additional discussion and design details on ramp metering.

EN-EN or EX-EX		EX-EN		Direct Connectors		EN-EX (Weaving)			
						 * Not Applicable to Cloverleaf Loop Ramps			
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.

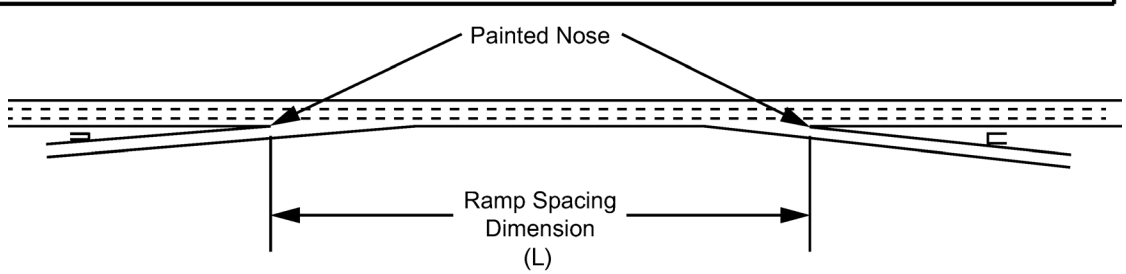


Figure 15-32: Arrangements for Successive Ramps

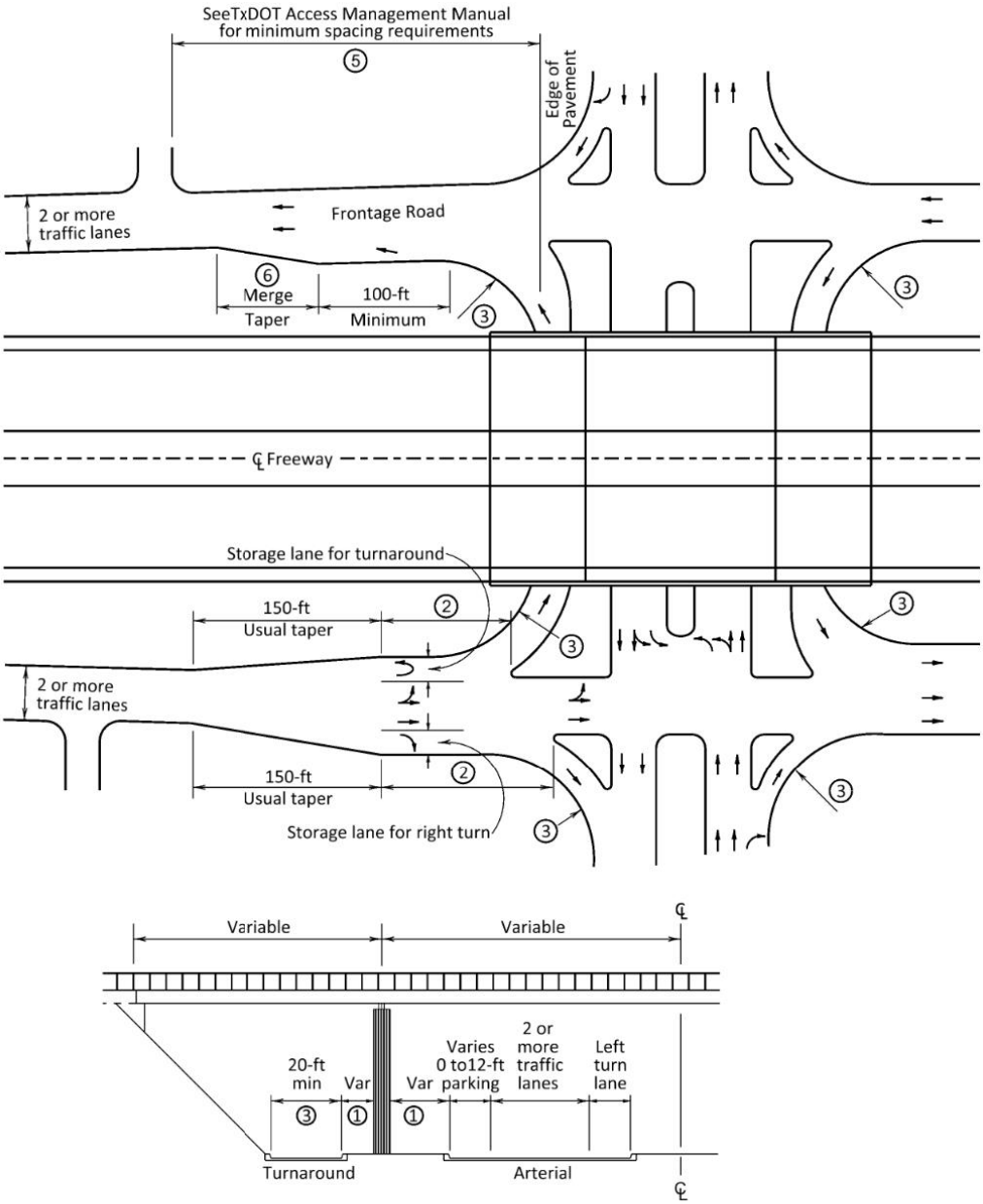
15.8 Frontage Road Turnarounds and Intersection Approaches

Turnaround lanes should 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.

The turnaround should be designed to accommodate speeds no more than 15 mph if possible. If lower or higher speeds are justified, a design exception is not required.

Figure 15-33 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 at least 525-ft in length. 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 15-3 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 15-33 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:

- 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 the target design vehicle as indicated in **Tables 5-1**, **6-1**, **7-1**, and **8-1**.
- 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 referred to in TxDOT's **AMM**.
- 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 turns are made from the through right lanes (i.e., no dedicated right turn lanes) and without acceleration lanes.
- 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.

- 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.

See **Section 18.5** for discussion on the design of bikeways in intersections. Additionally, refer to **TTI Study No. 06894** for additional information.

15.8.1 Guidelines for Designing Auxiliary Lanes (AL) on Frontage Roads Approaching Intersections

An auxiliary lane (AL) is a type of roadway lane that is built alongside of and typically adjoining to a roadway's primary lanes for a limited distance that varies depending on site needs and conditions. ALs are typically built to provide additional roadway capacity and help facilitate safe traffic movements such as speed changes and weaving, merging and diverging, entering and exiting, and turning. ALs are used on freeway frontage roads at and between ramp terminals and at intersections of the frontage roads with crossing roadways. They help balance the traffic load and provide transitions, vehicle storage, acceleration/deceleration to and from driveways/cross-streets, turnaround lanes, and interchange approaches and departures.

See **Section 8.1.8** for additional information regarding auxiliary lanes.

15.8.1.1 AL Merge Length

The minimum length of a frontage road merging AL should be 550', **not including the taper**. Where design (future year) volumes are expected to exceed 1,000 vph per lane or heavy vehicles are expected to exceed 10% of frontage traffic, the minimum length should be extended an additional 200', for a total AL length of 750'.

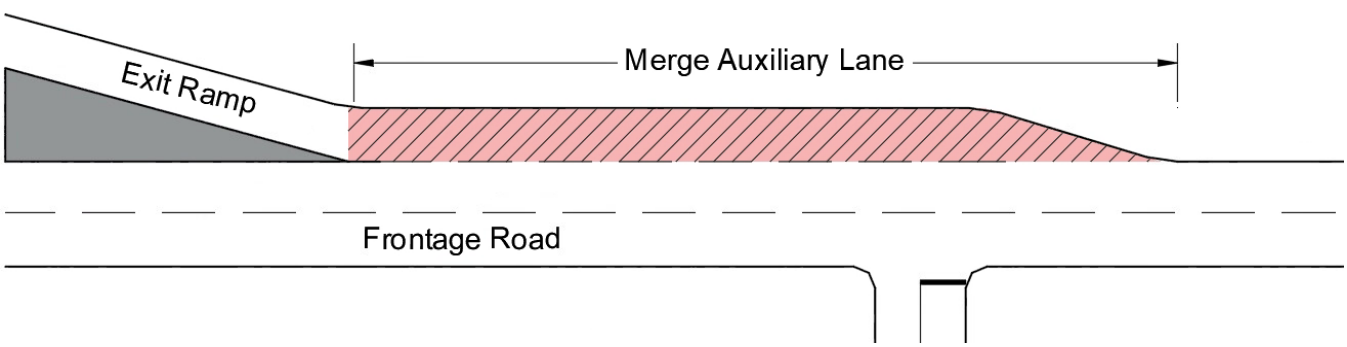


Figure 15-34 Example of a Merge Auxiliary Lane on a Frontage Road

15.8.1.2 X-Ramp Spacing

For X-Ramp configurations, simulation analysis and recent research suggest that the current frontage road minimum weaving section length of 1,000 feet established by the Green Book is applicable only in rural areas and less-developed urban areas without significant current or future frontage road development. However, in urbanizing and urbanized areas with development, the recommended frontage road weaving section length should be 1,500 feet, with a preferred length of 2,000'

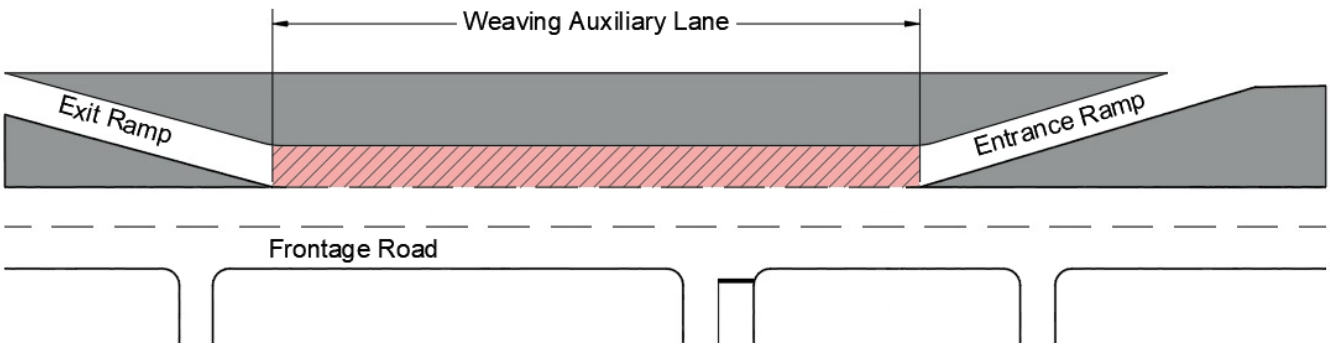


Figure 15-35 Example of a Weaving Auxiliary Lane on a Frontage Road

15.8.1.3 Distance from Exit Ramp to Downstream Intersection:

Fundamentally there are three distinct functional zones between the exit ramp/frontage road merge point and the stop bar for the intersection:

- A storage zone;
- A deceleration zone; and
- And a lane changing or weaving zone.

See **Figure 15-37** for a visual of these functional zones.

15.8.1.3.1 Queue Storage Zone

Findings confirm that queue lengths of about 180-ft are suitable for rural design and suggest that suburban and urban queue lengths should be even longer than the 420-ft suggested by previous research (**Tydlacka et al., 2017**). Queue lengths of at least 700 feet are appropriate for urban signalized intersections.

15.8.1.3.2 Deceleration Zone

The deceleration zone transitions to the storage zone as vehicles slow to a stop behind the longest, reasonably expected intersection queue under design year traffic conditions. Rural frontage roads typically have a higher operating speed than urban locations, so braking distances are greater outside of urbanized areas. See **Section 4.10.2** for additional information on deceleration length.

15.8.1.3.3 Weaving Zone

Through microsimulation and previous research **TTI Report 0-7047**, the minimum weaving zone for an urban context is 911.8 ft.,

and 548.2 ft. for rural context as shown in **Figure 15-36**.

When designing exit ramps in Texas freeway corridors with diamond interchanges, the minimum recommended spacing between the exit ramp frontage road gore and the downstream intersection is 1,750 ft in urban environments and 925 ft in rural areas. Rural operations improved and stabilized as ramp-to-intersection spacing increased from 500 to 1000 ft, and for urban areas at least 1,500 ft of total ramp-to-intersection spacing was necessary to minimize frontage road travel times/delay.

Context	Assumed Frontage Road Speed (mph)	Weaving Zone Length, Exit Ramp to Right Lane Change(s) (ft)	Deceleration Zone (ft)	Queue Storage Zone (ft)	Total Ramp-to-Interchange Spacing (ft)
Urban	45	911.8 (two lane changes)	117.6	700	1,750
Rural	55	548.2 (one lane change)	194.4	180	925

Figure 15-36: Comparison between Urban and Rural Weaving Zones

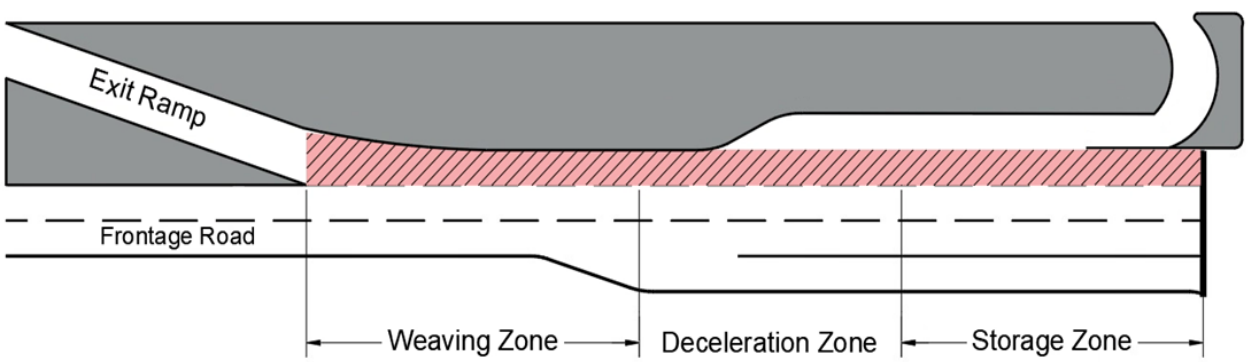


Figure 15-37: Distance from Exit Ramp to Downstream Intersection

15.8.1.4 Left-Turn Deceleration Lane Length

For a left-turn deceleration lane, the total desired length of a frontage road AL (left-turn lane, in this case) includes the sum of the entering taper, the deceleration distance, and the storage length, as shown in **Figure 15-38**. It also indicates that some deceleration is usually accepted within the through lanes – usually on the order of 10 mph – and that the taper is commonly considered part of the deceleration distance.

Left- and right-turn lanes also consider the taper length to be part of the deceleration length. This guidance provides a sound foundation for frontage road deceleration ALs functioning as intersection left- and right-turn bays as well as right-turn bays servicing driveways and minor side streets.

In rural environments, the 180-foot queue length check value is not long enough to cause blocking of left-turn lanes, since the lanes themselves will exceed this length using the design procedures contained in the RDM. However, for urban design, the left-turn lane length developed using the RDM should be checked against the 700-foot value from research **TTI Report 0-7047**. If the RDM-based left-turn length is less than 700 feet, the design length should be extended to at least 700 feet to ensure access to the left-turn lane is not blocked by a queue in the adjacent lane.

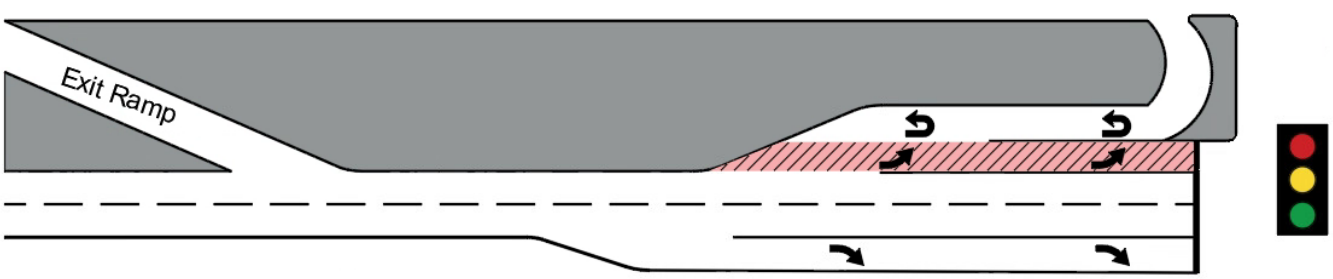


Figure 15-38: Left-Turn Deceleration Lane Length

15.8.1.5 Right-Turn Deceleration Lane Length

The 180-foot queue length check value for rural conditions is not expected to be an issue, since the RDM designed process will always produce a right-turn lane length longer than this value. However, in urban areas the 700-foot long check value should be compared against the RDM-derived right-turn lane length using traditional design procedures. If the urban right-turn lane length is less than 700 feet, extending the lane to at least 700 feet is recommended to lower the chance that access to the right-turn lane will be blocked by a queue in the adjacent lane. See **Figure 15-39** for a visual of the right-turn deceleration lane length.

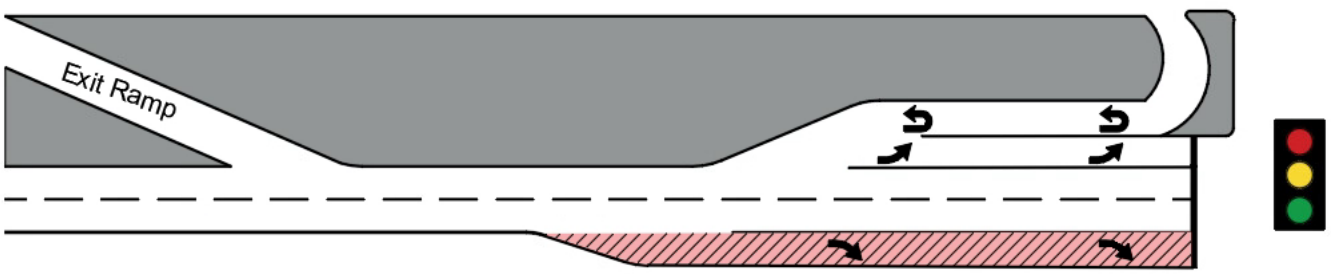


Figure 15-39: Right-Turn Deceleration Lane Length

- Rural Design: 180' adequate; and
- Urban Design: Recommended minimum 700' to avoid blocking issues.

15.8.1.6 U-Turn/Turnaround Deceleration Lane

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. **Overpasses in these areas should also be arranged to accommodate future turnarounds**. For a typical turnaround design for a diamond interchange, the minimum approach bay length should be 525 feet. However, for interchanges with high ADT (40,000 or greater) or that operate at LOS E or F, storage bay length should be determined through a traffic analysis.

When adjacent to left- and right-turn lanes (as shown in **Figure 15-40**), it is recommended that U-turn lanes in urban areas be at least 700 feet long so that access to left- and right-turn lanes is not blocked by a long queue in the adjacent left turn lane or left turn lane plus through lane at the intersection. Safety analysis of Texas U-turn lane interchanges indicate that longer U-turn lanes were associated with fewer crashes, but this effect is not statistically significant. A minimum length of 525-ft is recommended for U-turn lanes on rural and suburban frontage roads. For high-ADT intersections and/or those with LOS E or F, a traffic analysis should determine if the U-turn lane length is appropriate. The U-turn lane length should be at least 700 feet in length to avoid blocking issues, even if the analysis warrants a shorter length.

For additional information reference **TTI Research Report 0-7047 Establish Guidelines for Designing Auxiliary Lanes on Frontage Roads (2022)**.

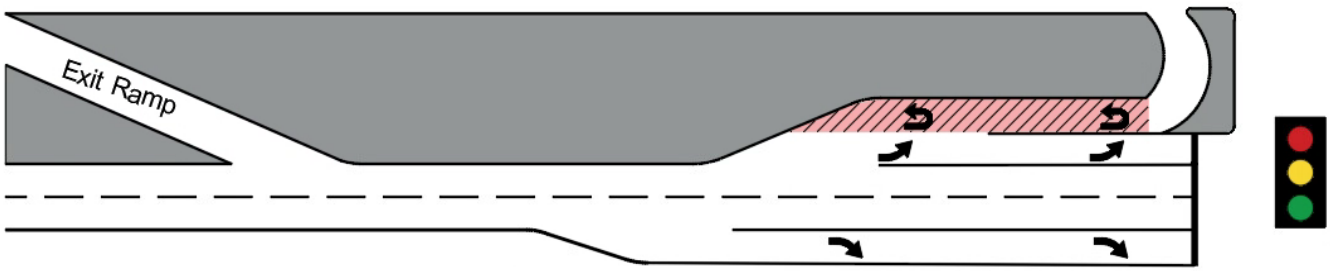


Figure 15-40: U-Turn/Turnaround Deceleration Lane

- Reasonable minimum length (525') recommended for rural and suburban frontage roads;
- Reasonable RDM recommendation to do a traffic analysis for high ADT intersections and/or those with LOS E or F; and
- **TTI Research Report 0-7047**: Recommended minimum 700' to avoid blocking issues.

Chapter 16
Driveways

16.1 Overview

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 consider 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 chapter applies to new driveways, and modification of existing driveways.

This chapter provides guidance on the location and geometric design of driveway connections intended to provide vehicular access to a highway on the State Highway System.

The guidance provided herein may not always be completely applicable due to the highly variable field conditions with respect to driveways.

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** unless the driveway is defined as a public driveway (see **Section 16.1.1** below).

However, departures from guidance may require support documentation in the form of traffic operations and safety analysis.

For additional information refer to TxDOT's **AMM** for permitting guidelines and for additional access discussion. For additional information on when **Intersection Control Evaluation (ICE)** may be considered at driveways refer to the **TSAP**, Section 10.2.2.

16.1.1 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 **43 TAC §11.51(1)**.
- **Commercial Driveway** - An entrance to, or exit from, any commercial business, or similar type establishment.
- **Divided Driveway** - A driveway providing a 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 (ROW), slightly raised curb (mountable) when U-turns are allowed or curbed (traversable) when U-turns are not allowed.

- **Driveway Apron** - Normally the distance from the roadway edge of pavement or face of curb and gutter to the back of taper/radius return.
- **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 ROW line may use lesser radii that fit within the State ROW**. See **Chapter 18** and **Chapter 19** for additional discussion on effective turning radius.
- **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 **43 TAC §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 **43 TAC §11.51(21)**. See **Section 4.15** for design requirements of Public Driveways.

- **Radial Return or Flared Taper Return** - The physical transition connection 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, wastewater 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 16-1**.
- **Throat Width** - The driveway width measured at the end of the return radii. Refer to **Figure 16-1**.

16.2 General Design Guidelines

The following guidelines apply to all driveways on a State Highway. **Figure 16-1** illustrates the driveway design elements including return radius, entry width, exit width, throat width, and throat length.

- The driveway placement should be such that drivers approaching from the main roadway will have sufficient sight distance to ascertain the driveway's location 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. See **Section 13.5** 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 **AMM**. When acceleration or deceleration lanes are provided, see **Section 4.10.2** for proper lengths, and recommended applications.
- 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 16-2** 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 Roadway Standards - Pedestrian Facilities Curb Ramps** must be provided.
- 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.
- Where a driveway is expected to cross a pedestrian or bicycle facility now or in the future, special design requirements may apply. See **Section 16.2.3, Chapter 18**, and **Chapter 19** for more information.

- 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 ROW.
- 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.

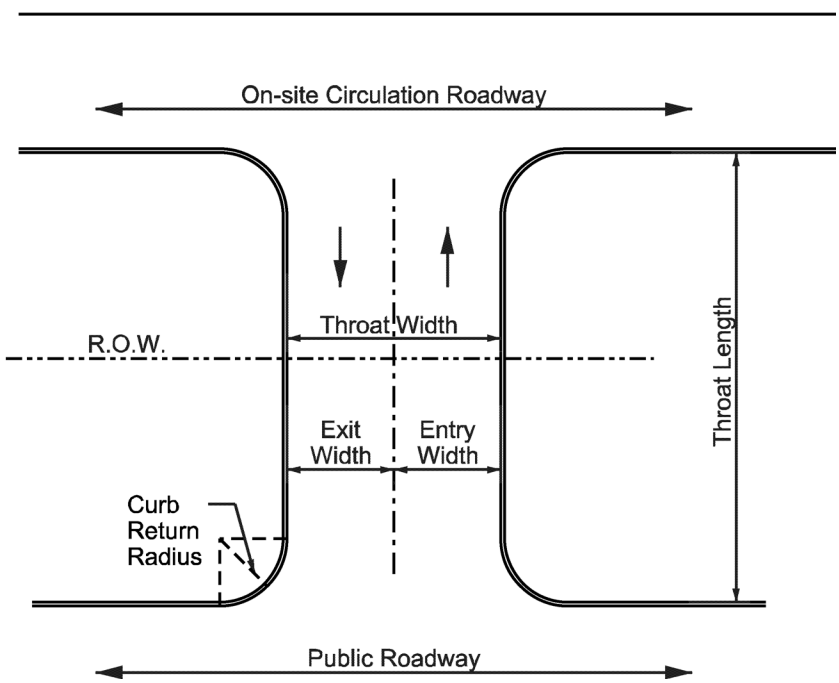


Figure 16-1: Driveway Design Elements

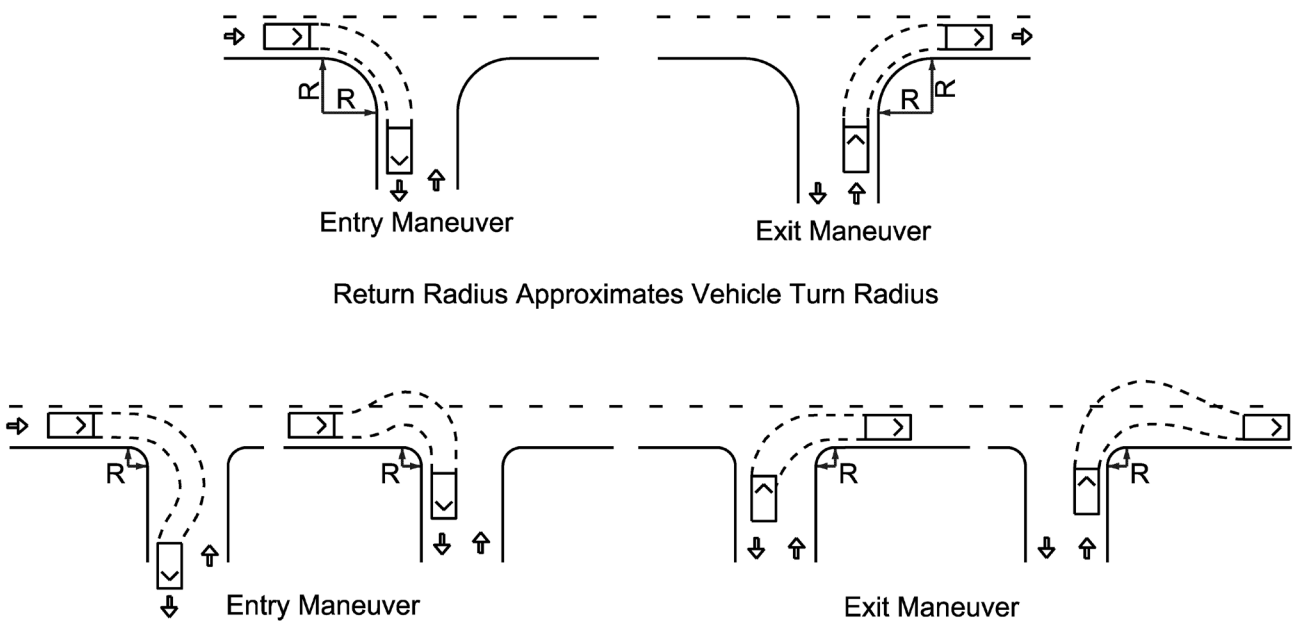


Figure 16-2: Effects of Return Radius on the Right-Turn Maneuver

16.2.1 Geometrics for Two-Way Driveways

The following are standards for two-way driveways. See **Figure 16-3** through **Figure 16-6** for examples of two-way driveways.

- **Private Residential Driveway** - normally designed as non-simultaneous two-way driveways. Standard design criteria for private residential driveways are provided in **Table 16-1**. If the throat width does not match the existing driveway at the ROW line and additional ROW is required, then match the existing driveway throat width to avoid unnecessary ROW acquisition to meet minimum throat width requirements. If additional length of driveway is needed, then an easement or similar should be obtained to connect to the driveway outside of TxDOT ROW.

Table 16-1: Design Criteria for Two-Way Private Residential Driveways

Radius[1] (ft)	Throat Width	
	Standard (ft)	Maximum (ft)
15	14	24

Notes:

1. Urban roadways with limited distance from the road ROW line may use lesser radii that fit within the state ROW. 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 16-2 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.

- **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.

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 appropriate 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 AMM for additional information.

Table 16-2: 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 Figure 16-3)	25	28
One entry lane and one exit lane, 4 or more SU vehicles[3] per day (see Figure 16-3)	30	30
One entry lane and two exit lanes, without divider (see Figure 16-4:)	25	40
One entry lane and two exit lanes, with divider (see Figure 16-5:)	25	44[1] - 55[2]
Two entry lanes and two exit lanes, with divider (see Figure 16-6:)	25	56[1] - 67[2]
Notes:		
1. See Table 16-3 for minimum divider widths. 2. See Table 16-3 for maximum divider widths. 3. Driveway designs for larger vehicles will be considered on a case-by-case basis.		

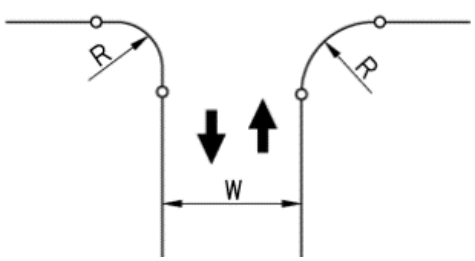


Figure 16-3: One Entry Lane/One Exit Lane

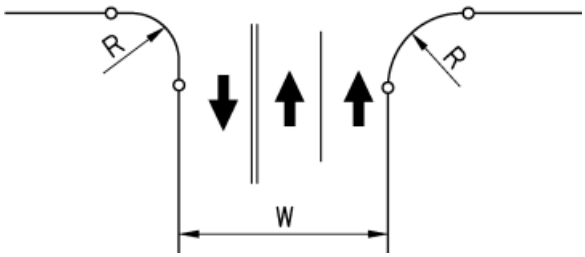


Figure 16-4: One Entry Lane/Two Exit Lane (Without a Divider)

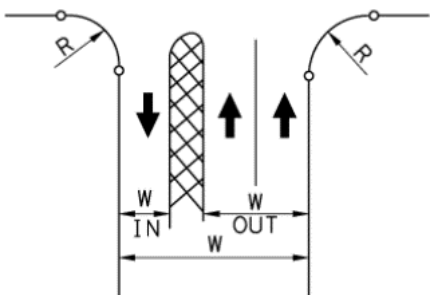


Figure 16-5: One Entry Lane/Two Exit Lanes (With a Divider)

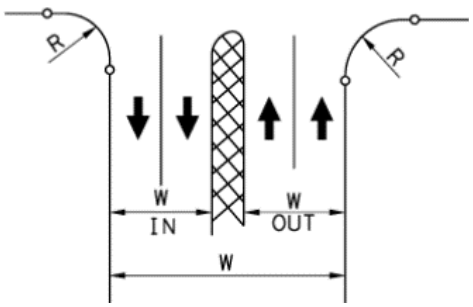


Figure 16-6: Two Entry Lanes/Two Exit Lanes (With a Divider)

16.2.2 Geometrics for 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 16-3**.

Table 16-3: Dimensions for Dividers in the Driveway Throat to Separate Entry and Exit Sides of the Driveway

Treatment	Width[3]	Length
Slightly raised[1] (4-in) with contrasting surface	4[2] –15	20
Notes:		
1. For Rural - Rounded edges, 30° to 45° slope. (See Figure 16-7). 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 16-7 illustrates a slightly raised divider (height ≥ 4 inches).

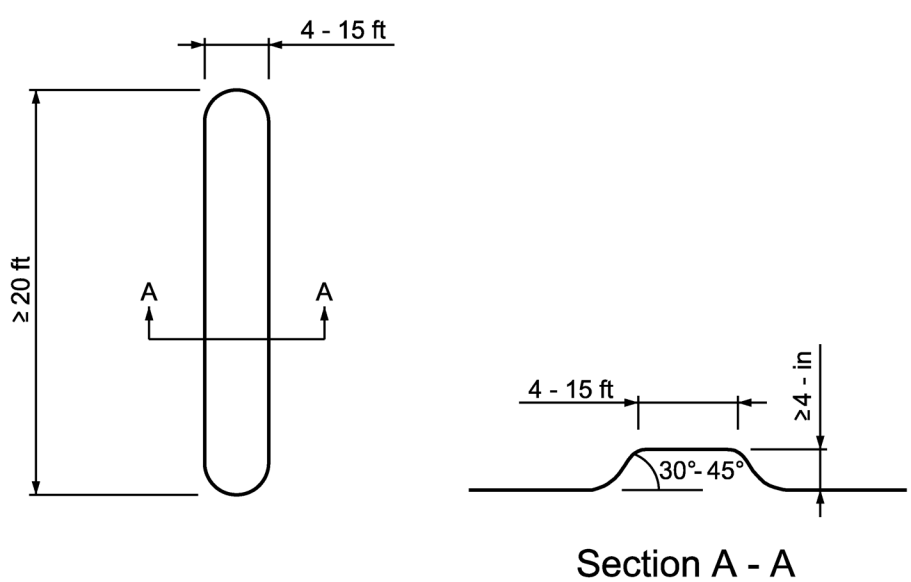


Figure 16-7: Illustration of Slightly Raised Divider

A divided driveway is desirable in the following situations:

- There is 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 (refer to the **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**.

16.2.3 Bicycle and Pedestrian Considerations

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). See **Chapter 18** and **Chapter 19** for more information.

16.3 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.

16.3.1 Driveway Grades

To achieve satisfactory driveway profiles, some of the significant factors to be considered are:

- 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.
- The comfort of vehicle occupants and potential vehicle damage (i.e., prevent the dragging of center or overhanging portion of passenger vehicles), and;
- The compatibility of grades with the site requirements for sight distance and drainage to prevent excessive drainage runoff from entering the roadway or adjacent property.

Where a driveway is expected to cross a pedestrian or bicycle facility now or in the future, special design requirements may apply. See Chapter 18 and Chapter 19 for more information.

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

within 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 16-4**, 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.

Construction practice can provide a suitable sag vertical curve between the pavement cross-slope and the driveway apron when the apron length La (see **Figure 16-8**) is greater than or equal to 20 ft.

Where possible, the driveway grade should be limited to 6 percent or less within the roadway ROW. Maximum driveway grades should be limited to 12 percent for private residential driveways and to 8 percent for other driveways. When suggested maximum grades are not attainable due to terrain conditions, 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.

Table 16-4: 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.	

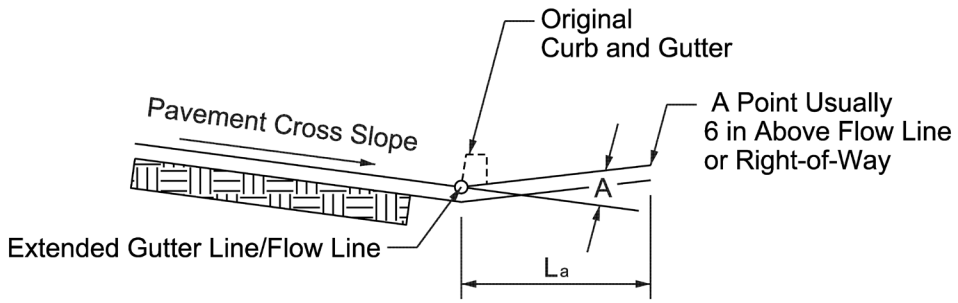


Figure 16-8: Suggested Dimensions to Achieve an Appropriate Vertical Curve

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 16-5** provides the desirable and minimum lengths for these vertical curves when used.

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 16-6** provides the typical lengths for these vertical curves.

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). Larger K-values may be needed for specialized vehicle uses.

See **Table 16-5** and **Table 16-6** for lengths of vertical curves.

A construction easement or license agreement is required for driveway construction beyond the ROW line.

For construction beyond the ROW, it is necessary for the property owner to furnish the construction easement or right of entry required.

Table 16-5: Length of Vertical Curve, L, For a Change in Grade Between the Pavement Cross-Slope and the Driveway Apron Slope

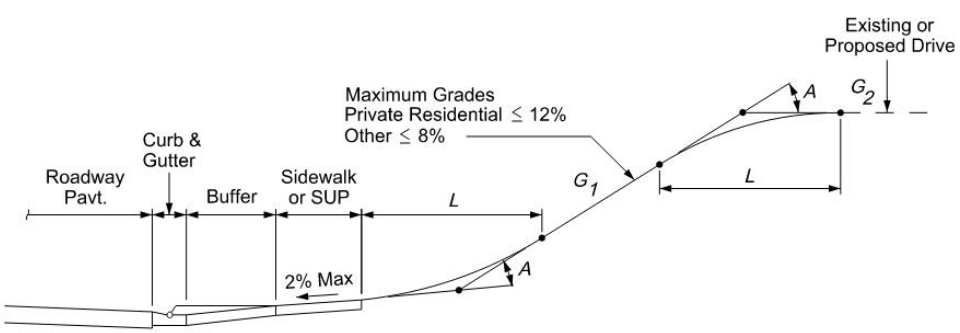
Change in Grade, A	Crest		Sag	
	Des.	Min.	Des.	Min.
	ft	ft	Ft	ft
4-5%	5	3	7	4
6-7%	6	4	8	5
8-10%	8	5	10	7

Table 16-6 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

16.3.2 Profiles on Curb and Gutter Sections

Figure 16-9 and **Figure 16-10** illustrate typical driveway profiles.



Buffer Between Sidewalk and Curb

G = Grade (%)
A = Algebraic Difference in Grades (%)
L = Min. Length of Vertical Curve

Figure 16-9: Roadway with Curb and Gutter, Driveway Profiles on an Upgrade

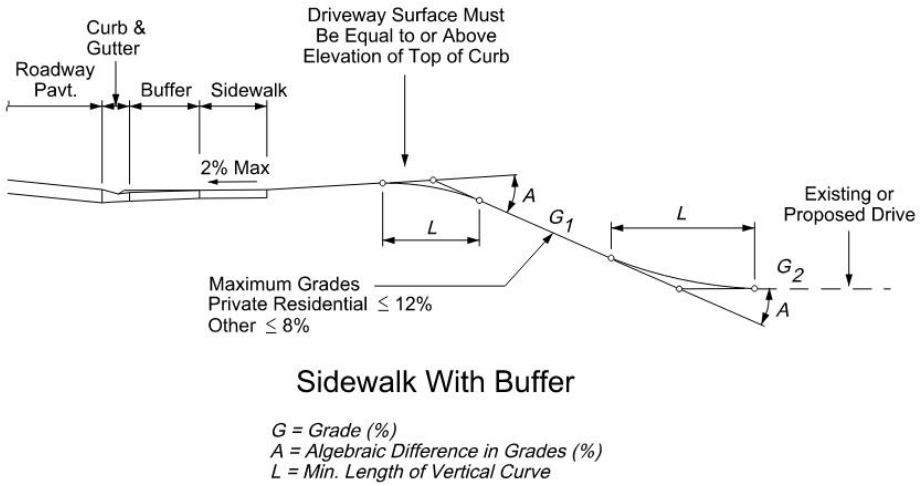


Figure 16-10: Roadway with Curb and Gutter, Driveway Profiles on a Downgrade

16.3.3 Profiles with Drainage Ditch

Figure 16-11 illustrates the typical driveway profile.

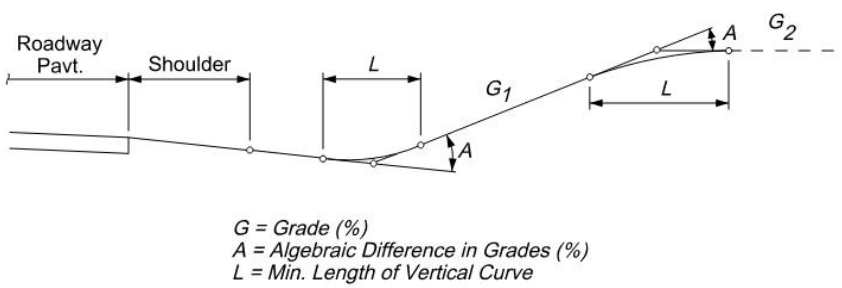


Figure 16-11: Driveway Profiles on Roadway with Drainage Ditch

16.4 Driveway Angle

16.4.1 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 driveway 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; and
- **Public Driveway, Service Driveway and Field Driveway** - 80 degrees.

16.4.2 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).

16.5 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. Consider the sight distance for vehicles entering and exiting the driveway as well as cyclists and pedestrians crossing the driveway in placement of signs. **Private signs are prohibited on State ROW**.

Chapter 17
Roadside Safety Design and Roadside Safety Hardware

17.1 Overview

This chapter provides guidance on the use of safety treatments that minimize the likelihood of serious injuries when a driver runs off the road.

The roadside is defined as that area beyond the traveled way (i.e., driving lanes) and the shoulder (if any) of the roadway itself. Consequently, roadside delineation, shoulder surface treatments, and similar on-roadway safety features are not extensively discussed.

This chapter includes the following chapters and subsections:

17.1 Overview

17.2 Roadside Safety Hardware Crash Criteria

- 17.2.1 MASH Background; and
- 17.2.2 Current TxDOT MASH Implementation Timetable/Policy.

17.3 Longitudinal Barriers

- 17.3.1 Types of Barriers;
- 17.3.2 Applications; and
- 17.3.3 Working Width.

17.4 Concrete Barriers (Median and Roadside)

- 17.4.1 Application;
- 17.4.2 Location; and
- 17.4.3 Standard Installations.

17.5 Guard Fence

- 17.5.1 Blockouts;
- 17.5.2 Structural Considerations of Guard Fence;
- 17.5.3 Placement of Guard Fence; and
- 17.5.4 End Treatment of Guard Fence.

17.6 Determining Length of Need of Barrier

- 17.6.1 Variables;
- 17.6.2 Design Equations;
- 17.6.3 Using Design Equations to Determine Length of Guard Fence; and
- 17.6.4 Example Problems.

17.7 Median Barrier

- 17.7.1 Application; and
- 17.7.2 Placement.

17.8 Attenuators (Crash Cushions)

- 17.8.1 Application;
- 17.8.2 Crash Cushion Categories;
- 17.8.4 Location; and
- 17.8.5 Standard Installations.

17.9 Emergency Crossovers

- 17.9.1 Location; and
- 17.9.2 Construction.

17.2 Roadside Safety Hardware Crash Criteria

AASHTO's Manual for Assessing Safety Hardware (MASH) 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 Report 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.

17.2.1 MASH Background

The November 20, 2009 memorandum from David A. Nicol specified the **AASHTO's Manual for Assessing Safety Hardware (MASH)** as the latest in roadside safety hardware criteria. 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. Additional relevant memoranda are as follows:

- 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**. 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.

- The AASHTO/FHWA Joint Policy Resolution PR-1-23 effectively ended the 2016 AASHTO/FHWA Joint Implementation Agreement.

17.2.2 Current TxDOT MASH Implementation Timetable/Policy

As product manufacturers and developers have continued 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, and Mailbox Standards 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 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 - Project Delivery Section Roadway Standards and Research Group webpage**.

The repair and replacement policies of Roadside Safety Hardware are governed by TxDOT's **Maintenance Operations Manual** and the applicable **TxDOT Specifications**.

17.3 Longitudinal Barriers

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; and
6. Delineate the obstacle if the above alternatives are not appropriate.

17.3.1 Types of Barriers

17.3.1.1 Rigid

Common rigid barriers used by TxDOT are constant single-slope barriers and F-shape barriers that are cast in place or precast barriers anchored in accordance with TxDOT cast in place barrier standards. 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 less than 1-ft of deflection is anticipated for a TL-3 (high-speed) pickup truck impact. Where considerable truck traffic exists (10% or greater) 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 in narrow medians to limit deflection and reduce the need for maintenance and repair. The exposed ends of the barrier need to be protected by an attenuator.

17.3.1.2 Semi-Rigid

Semi-rigid barriers commonly used by TxDOT include metal beam guard fence and both pinned and unanchored (a.k.a., freestanding) precast concrete barriers. Semi-rigid barriers have an expected deflection between 24 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 Midwest Guardrail System (MGS) 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 non-traversable 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 and motorists from work zone obstacles.

17.3.1.3 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 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.

17.3.1.4 Barrier Transitions

When transitioning from one type of barrier to another with significant differences in deflection or geometric shape 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 Semi-rigid (metal beam guard fence) to Rigid barriers available on the **TxDOT Roadway Standards** webpage. The purpose of all these transitions is to gradually decrease the expected deflection to safely redirect an errant vehicle.

17.3.1.5 Special Barrier Applications

In the application of traffic control plans, situations may arise where transitions are needed from a rigid 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. Refer to 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 DES, Project Delivery Section – Roadway Standards and Research Group.

17.3.2 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 17-1: shows a summary of roadside features that are commonly shielded with guard fence. The abbreviation “cz” means within clear zone.

Table 17-1: General Applications of Conditions for Roadside Barriers

Roadside Feature	Applications
Terrain Features:	
Steep Embankment Slope	cz[1], See Figure 17-1
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]
Quarries and Pits	see chapter 24 section 7
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 Section 4.12.7.
3. For specific 4R requirements see Section 4.10; for 3R see Section 10.3.

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 (i.e., 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. See **Section 4.10.7** for additional information.

The combination of embankment height and side slope rate may indicate barrier protection consideration as shown in **Figure 17-1**. 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 slopes steeper than 1V:4H.

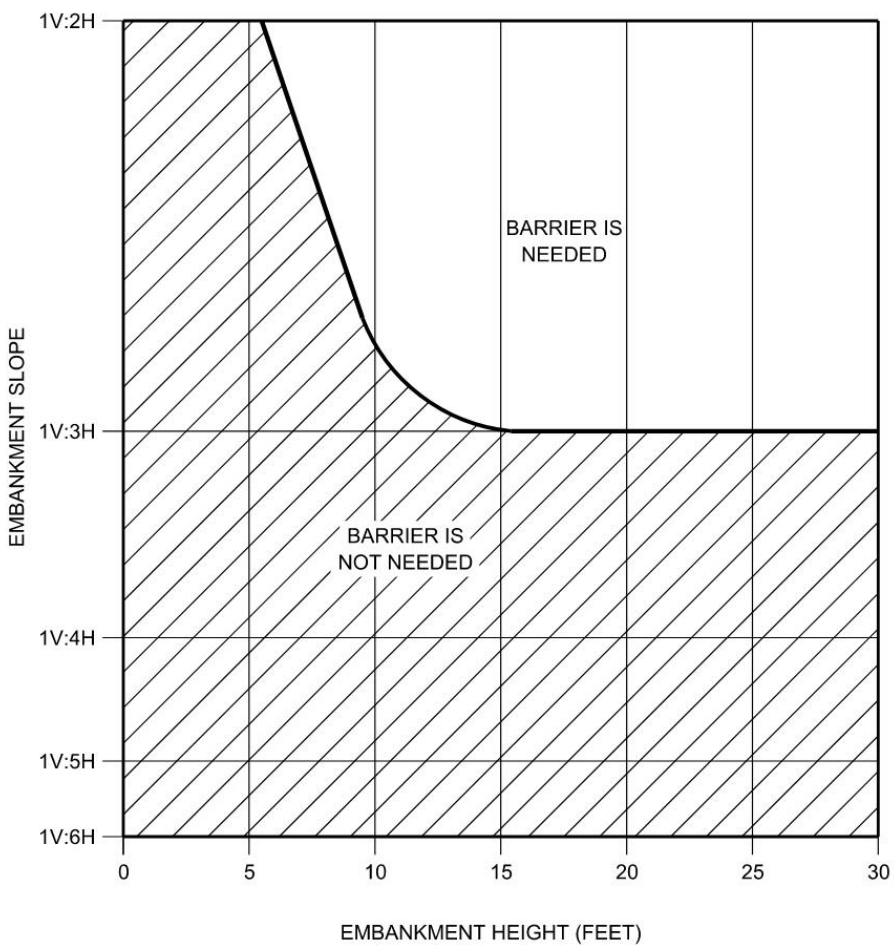


Figure 17-1: Guide for Use of Barrier for Embankment Heights and Slopes

17.3.3 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 during the impact (see **Figure 17-2**). Working width is related to deflection but working width considers the lateral position of the vehicle or any portion of the system. Working width should be considered when placing any longitudinal barrier. The Working height is the corresponding height at the maximum lateral position of any major part of the system or vehicle.



Figure 17-2: Working Width

17.4 Concrete Barriers (Median and Roadside)

17.4.1 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.

17.4.2 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 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.

17.4.3 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 **Section 4.10.15**) 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 impact.

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.

17.5 Guard Fence

Guard fence 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. Guard fence is designed to prevent impacts with roadside obstacles by redirecting the vehicle so that it continues to move at a reduced velocity in the original direction of traffic. The limits of guard fence 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 guard fence applications. Refer to the respective TxDOT guard fence standards for specific guidance.

Guard fence should be installed in accordance with current roadway standards.

17.5.1 Blockouts

The guard fence is blocked out from the posts with routed timber or composite blockouts (nominally 6-in x 8-in x 14-in). These blockouts reduce vehicle snagging on the posts and 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 in depth) to increase the post offset to avoid obstacles such as utilities. 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 offset) for one post for every 75-ft of guard fence.

17.5.2 Structural Considerations of Guard Fence

Post spacing, rail shape and thickness, rail height, splice strength and location, post embedment, and rail anchorage are all key factors that influence the structural integrity of guard fence.

17.5.2.1 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 there should be no slope break within 2-ft beyond the back of the post location as shown in **Figure 17-3**, 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 guard fence.

Embedment depth is shown on the standard detail sheet for both timber and steel posts.

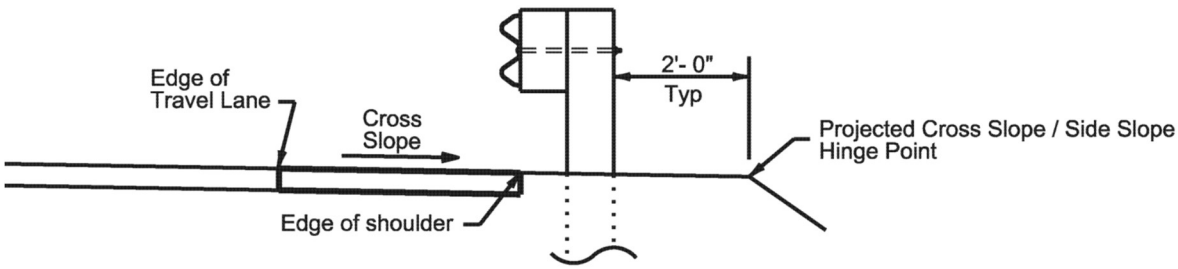


Figure 17-3: Lateral Support to Accommodate Guard Fence

17.5.2.2 Rail Element

Guard fence is fabricated in a w-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. The standard length of the rail element is 25-ft, or 12.5-ft.

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 must be spliced midspan between the posts.

Pavement overlays effectively reduce existing rail height. The following guidelines must be followed:

- When rail height varies more than 3-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 instability or vaulting or under riding the system;
- For existing 28-in rail systems, the rail height must 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 (when splice is at the post location), 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; and
- 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 angle strut locations.

17.5.3 Placement of Guard Fence

Guard fence 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.

The placement of guard fence pertains to the lateral and longitudinal position.

17.5.3.1 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 17-4**.

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. For guidance in possible adjustments with respect to guard fence in the vicinity of curbs contact DES – Project Delivery Section.

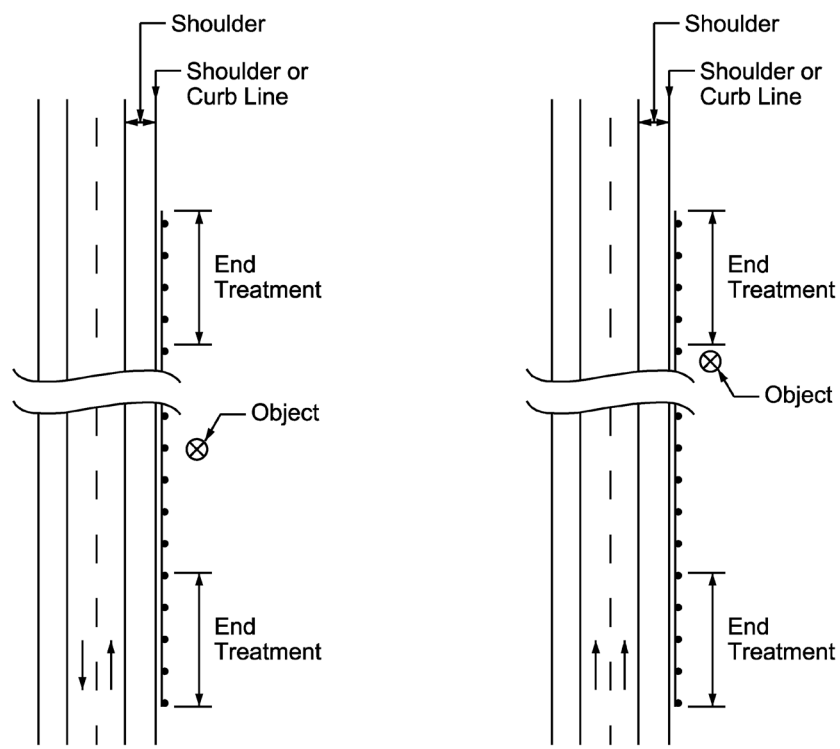


Figure 17-4: Placement at Shoulder Edge or Curb Face

17.5.3.2 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. Guard fence placement guidance is shown in **Figure 17-5**. 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. Also, there should be no slope break within 2-ft beyond the back of the post location.

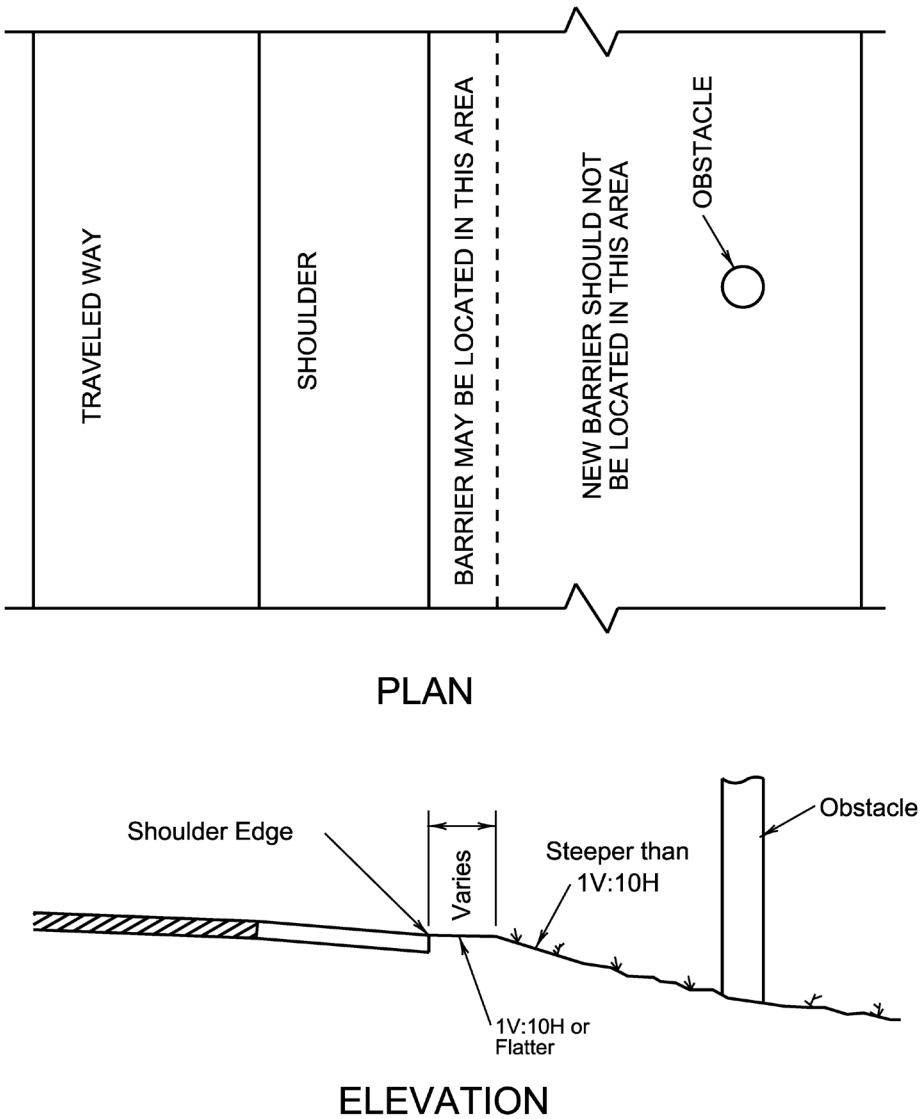


Figure 17-5: Location of Roadside Guard Fence

17.5.3.3 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 17-6**. **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. Refer to **Roadside Safety Pooled Fund Test Report No. 610211-01 (October 2021)** for specific guidance.

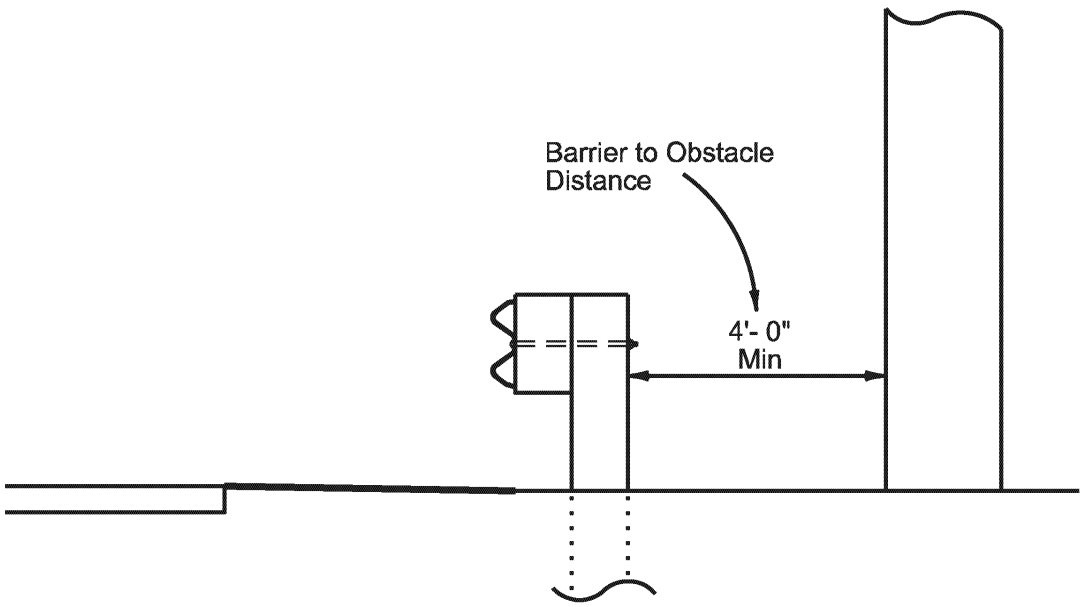


Figure 17-6: Allowance for Deflection of Guard Fence

17.5.4 End Treatment of Guard Fence

Providing appropriate end treatments is one of the most important considerations in the design of guard fence. An untreated guard fence will stop a vehicle abruptly and can penetrate the passenger compartment.

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 and downstream 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 17-4**). In that case a Downstream Anchor Terminal (DAT) end treatment without offset is acceptable for use.

17.6 Determining Length of Need of Barrier

The shape of the obstacle, its location with respect to travel lanes, the volume of traffic, its corresponding clear zone width, and the lateral location of barrier 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.

17.6.1 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 17-7**, will be referred to as an "area of concern."

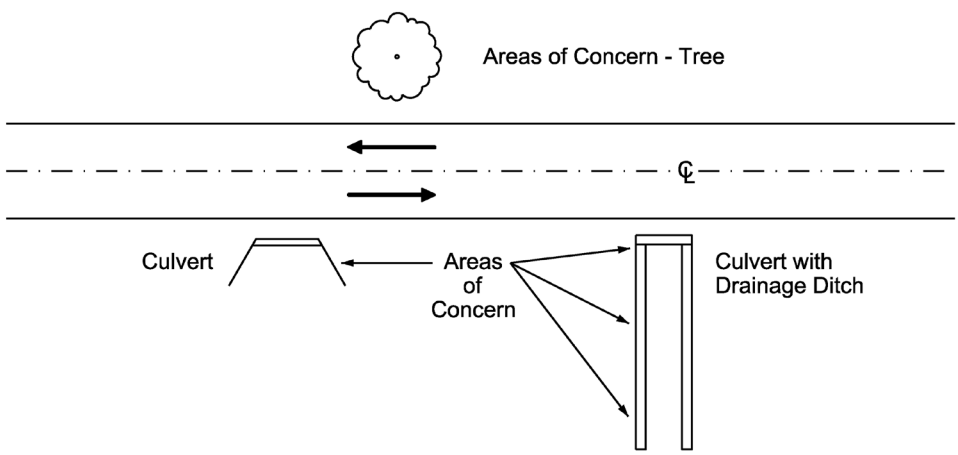


Figure 17-7: Areas of Concern

Figure 17-8 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:

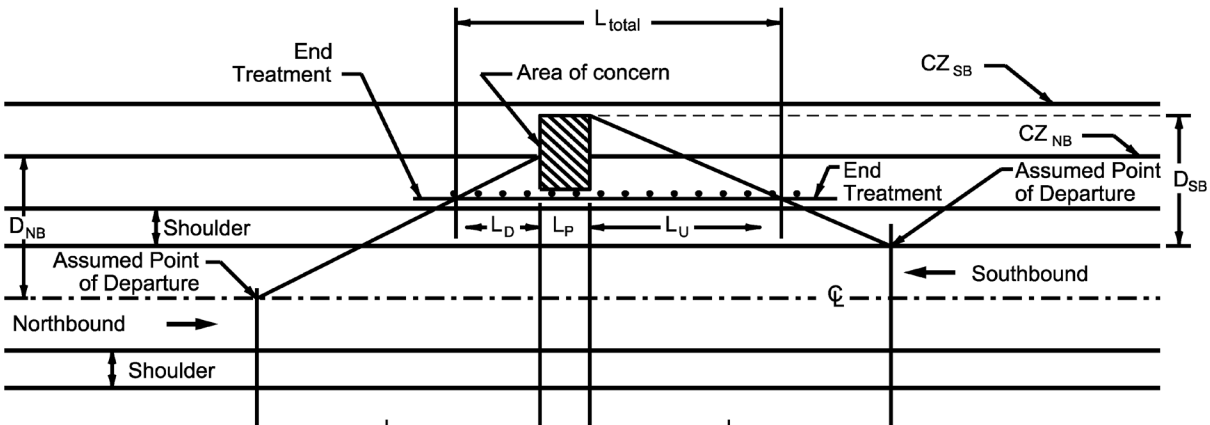
$$L_{total} = L_u + L_p + L_d \qquad \textbf{Equation 17-1}$$

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 shield the area of concern for 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 17-8**) traffic as

measured from the centerline pavement markings.



CZ = Clear Zone Width

CZ_{SB} = Clear Zone Width for Southbound Traffic

CZ_{NB} = Clear Zone Width for Northbound Traffic

L_{RT} = Runout Length required to shield obstacle

D_{NB} = Distance from edge of northbound travel lane to far side of area of concern or to outside edge of clear zone, whichever is least.

D_{SB} = Distance from edge of southbound travel lane to far side of area of concern or to outside edge of clear zone, whichever is least.

L_U = Guard Fence Length Upstream of Area of Concern

L_P = Guard Fence Length Parallel of Area of Concern

L_D = Guard Fence Length Downstream of Area of Concern

L_{total} = Length of Guard Fence Needed (L_U+ L_P+ L_D)

Figure 17-8: 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 when 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 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 either inside or outside the clear zone, D_u equals the clear zone distance. Additionally, D_u equals the clear zone distance for any bridge structure.

17.6.2 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 17-1** for calculating the guard fence length of need.

Table 17-2: Equations for Upstream and Downstream Length of Need

ADT ≤ 750	$L_u = 200 - \frac{200}{D_u} x G_u$	**Equation 17-2**
	$L_d = 200 - \frac{200}{D_d} x G_d$	**Equation 17-3**
ADT > 750	$L_u = 250 - \frac{250}{D_u} x G_u$	**Equation 17-4**
	$L_d = 250 - \frac{250}{D_d} x G_d$	**Equation 17-5**

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)

For high volume conditions, if the clear zone width (30-ft) is met or exceeded, L=0.

17.6.3 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);
- Approach grading, (i.e., will slopes be 1V:10H or flatter);
- Placement location, lateral offset measured to barrier face (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 17.6.4 provides example problems and solutions using the design equations.

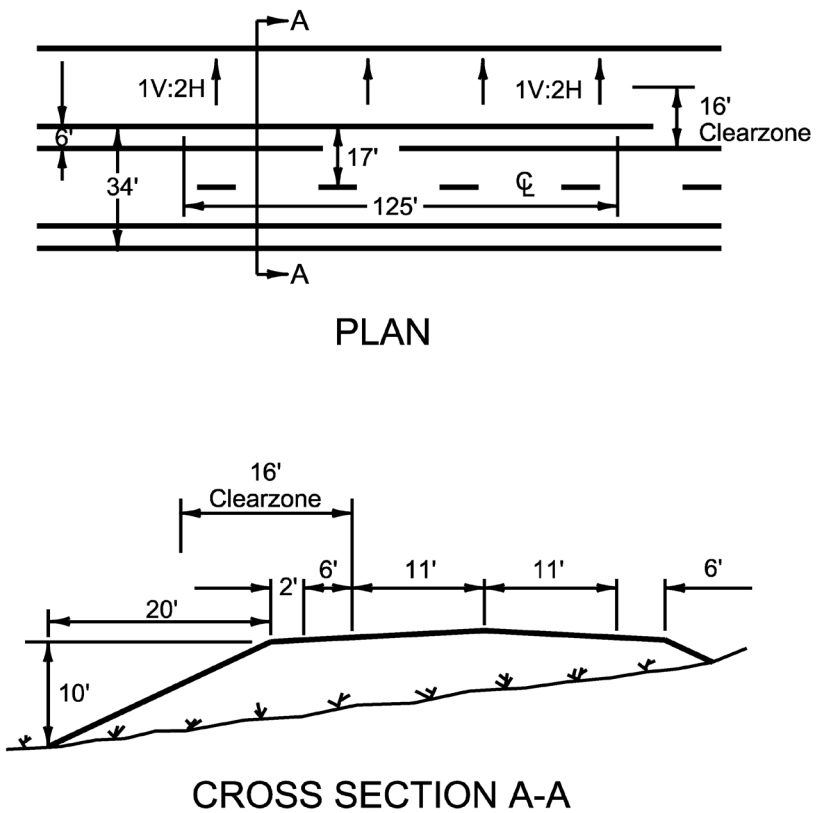
The guard fence lengths produced by the equations should be rounded up to an even length of guard fence.

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.**

17.6.4 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 17-9**. 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.



EXAMPLE 1 PROBLEM LAYOUT RURAL LOW VOLUME

Figure 17-9: Example 1 Problem Layout Rural Low Volume.

Solution: From the information above and referring to **Figure 17-9** it is determined that a "rail is needed." As shown in **Equation 17-1**, 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, **Equation 17-2** and **Equation 17-3** are used to solve for L_u and L_d, respectively (if necessary).

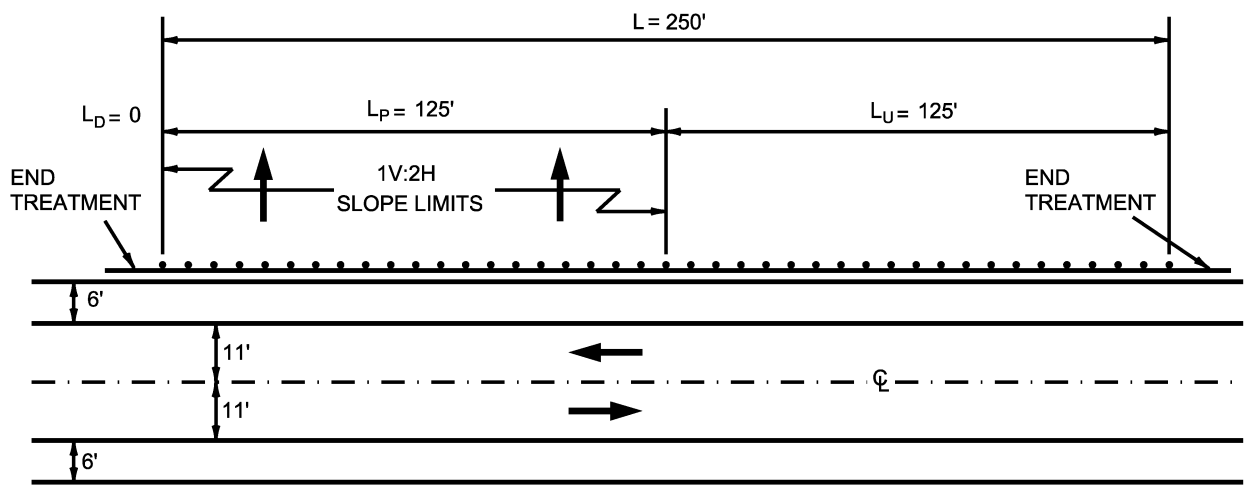
For the upstream direction, the area of concern is the full (16-ft) clear zone width and the guard fence offset (G_u) is 6-ft. Substituting in **Equation 17-2**.

$$Lu = 200 - \frac{200}{16} * 6 = 125 \text{ ft}$$

A placement of guard fence alongside the 6-ft wide shoulder results in L_u = 125-ft.

Referring to **Figure 17-9**, 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 17-10** 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.

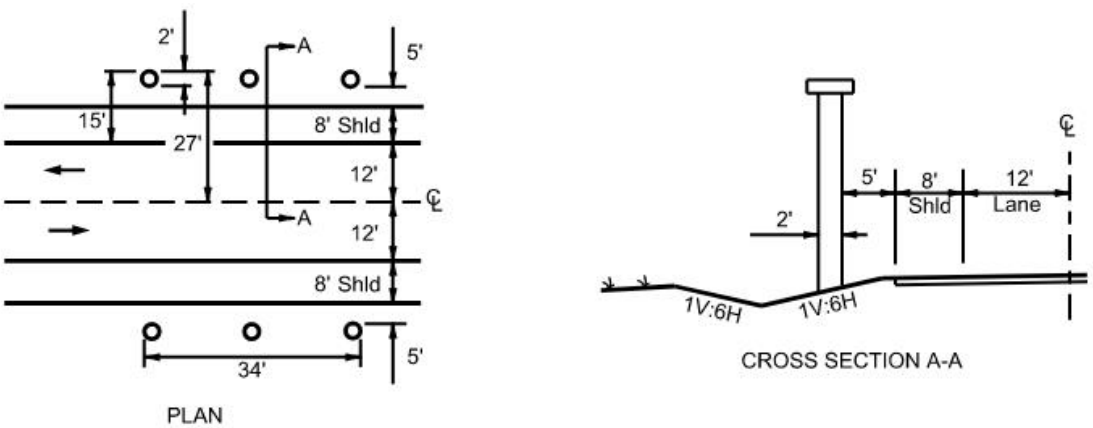


EXAMPLE 1 PROBLEM SOLUTION GUARDRAIL LAYOUT

Figure 17-10: 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 17-11**. The areas of concern are bridge bents located 5-ft from the edge of shoulder. The side slopes are 1V:6H.



EXAMPLE 2 PROBLEM LAYOUT RURAL HIGH VOLUME

Figure 17-11: Example 2 Problem Layout Rural High Volume

Solution: (Note, North is assumed to be on the top side of Plan view). Referring to **Table 17-1** 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 17-11**. As shown in **Equation 17-1** the length of need is $L_{total} = L_u+L_p+L_d$. L_p is 34-ft from the given information. Because the ADT is greater than 750, **Equation 17-4** and **Equation 17-5** are used to find L_u and L_d, respectively:

$$L_U = 250 - \frac{250}{15} * 8 = 116.5 \text{ ft}$$

Substituting in the equation, the upstream length (L_u) is 116.5-ft if placement is at the shoulder edge.

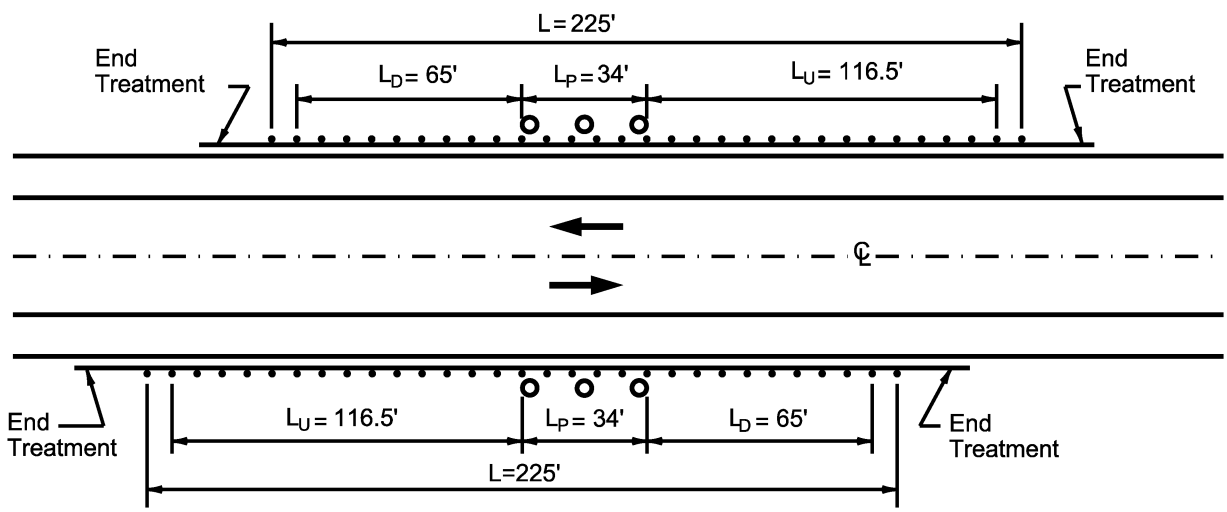
The downstream (westbound traffic) length of guard fence is determined by substituting into **Equation 17-5**:

$$L_D = 250 - \frac{250}{27} * 20 = 65 \text{ ft}$$

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 17-12** .

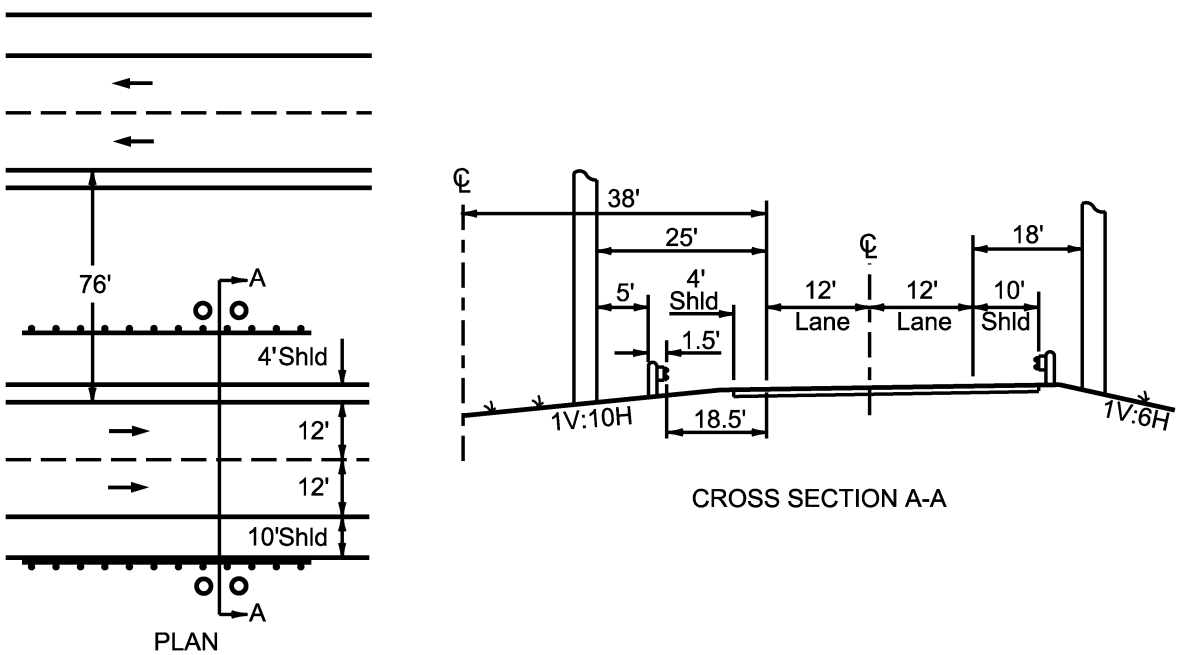


EXAMPLE 2 PROBLEM SOLUTION GUARDRAIL LAYOUT

Figure 17-12: 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 17-13**. 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.



EXAMPLE 3 PROBLEM LAYOUT DIVIDED HIGHWAY

Figure 17-13: 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 17-13**, guard fence may be placed thereon (see **Figure 17-5**). Therefore, place the guard fence such that the back of the posts is 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 17-1**, $L_{total} = L_u+L_p+L_d$. For one-way traffic operations, $L_d=0$; furthermore, for the overhead sign bridge support $L_p=0$. **Equation 17-4** is used to find L_u because ADT is greater than 750:

$$L_U = 250 - \frac{250}{25} * 18.5 = 65 \text{ ft}$$

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 (see **Figure 17-5**). Substituting in **Equation 17-4** to determine L_u:

$$L_U = 250 - \frac{250}{18} * 10 = 111 \text{ ft}$$

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 right side, placement is as shown in **Figure 17-14**.

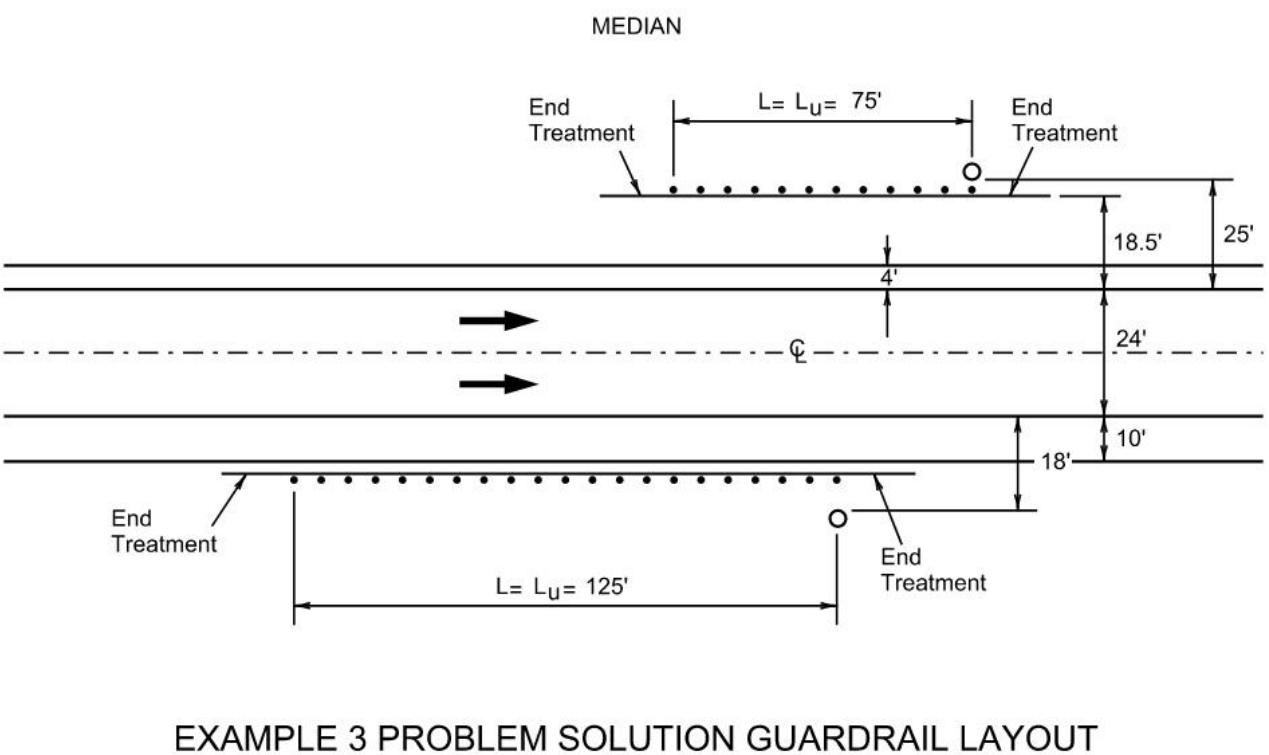


Figure 17-14: Example 3 Problem Solution Guardrail Layout

17.7 Median Barrier

Median barriers used to reduce median crossover vehicle encroachments 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 previous sections).

17.7.1 Application

On high-speed highways (50 mph or greater), median barriers should be considered based on the criteria shown in **Table 17-3**. Flush medians or frequent crossovers may preclude the use of median barriers based on an engineering analysis of individual locations.

Table 17-3: Recommended Guidelines for Installing Median Barriers on High-Speed Highways

Recommended Guidelines for Installing Median Barriers on High-Speed Highways

Median Width (ft)	AADT in 1000's																						
	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50	50-55	55-60	60-65	65-70	70-75	75-80	80-85	85-90	90-95	95-100	100-105	105-110	110-115	115-120	120-125
0-5	Barrier Recommended																								
5-10																									
10-15																									
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75-80																									
80-85																									
85-90																									
90-95																									
95-100																									
> 100	Evaluate Need and Cost Effectiveness of Continuous Barrier. (Point obstacle protection may be appropriate for specific locations)																								

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 (10% or greater);
    - 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 (e.g.; overhead sign bridges, etc.), or in areas of lesser concern (e.g., wider medians, forested areas, etc.).

17.7.2 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.

17.7.2.1 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 17-15**. 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 DES 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 17-15 shows examples of barrier placement in non-level medians.

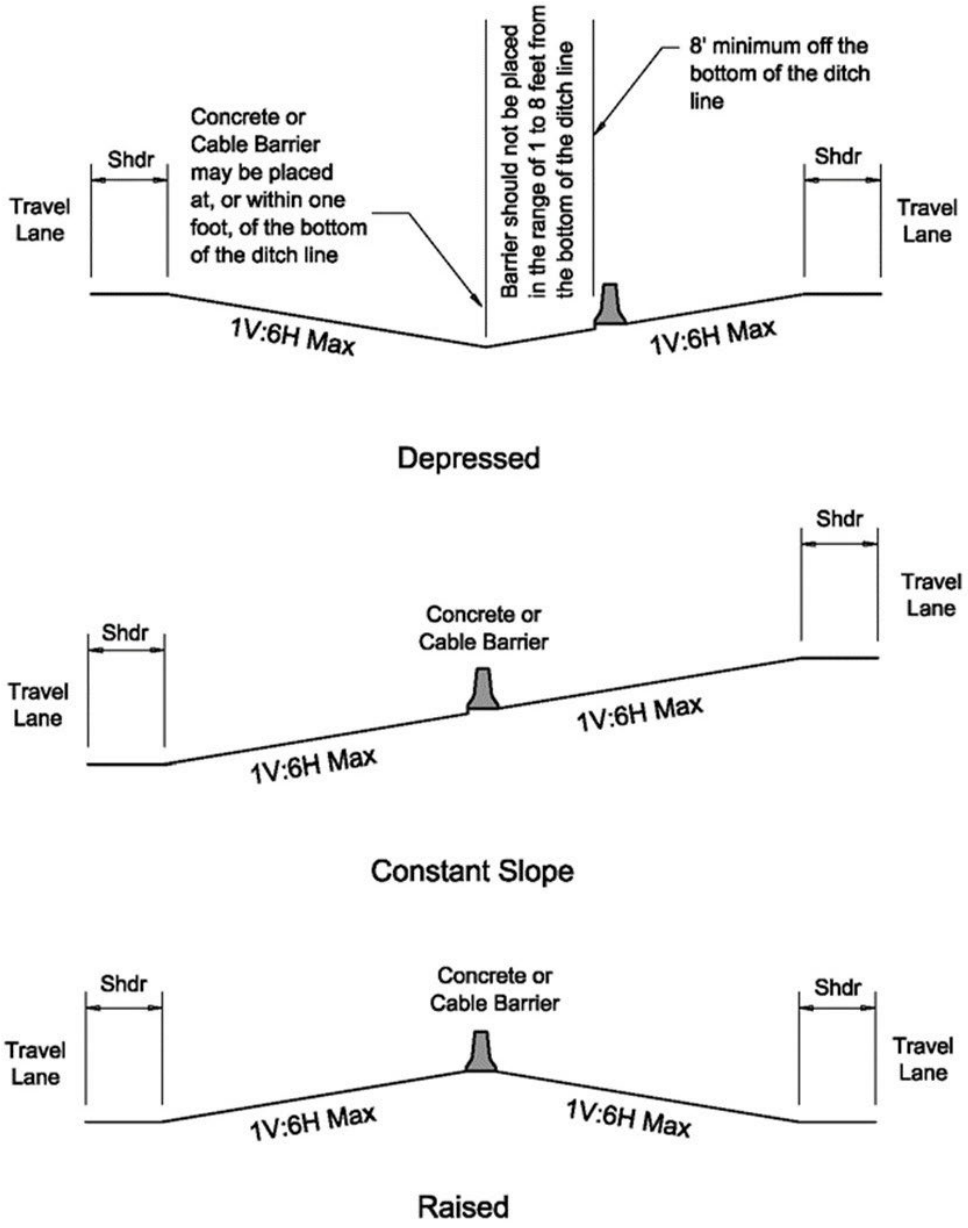


Figure 17-15: Barrier Placement in Non-Level Medians

17.7.2.2 Additional Cable Median Barrier Guidance

Cable barrier used in median applications is for use only 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. Note that cable barrier designated as TL-4 has been tested for that condition on flat, level ground. When placed on slopes the TL-4 performance may not be

achieved. 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 **NCHRP Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features** or the updated testing procedures found in **AASHTO's Manual for Assessment of Safety Hardware (MASH)**. Safety features approved to a minimum TL-3 are acceptable for low-speed and high-speed arterial highways. 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 5-ft 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 as shown in **Figure 17-16**. Reduced post spacing through these curves is recommended to minimize the length over which this occurs as shown in **Table 17-4**.

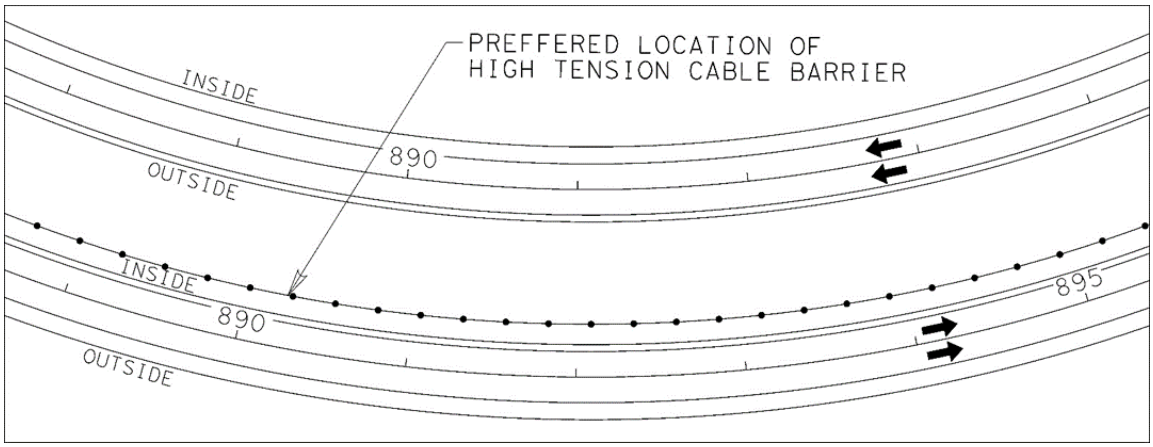


Figure 17-16: Desirable Cable Barrier Placement in a Curve

Table 17-4: Example 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

* See specific Manufacturers guidance for additional recommendations.

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. See specific Manufacturers guidance for allowable minimum length runs.

17.8 Attenuators (Crash Cushions)

17.8.1 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.

17.8.2 Crash Cushion Categories

Crash cushions are classified in one of three categories based on repair costs, lifecycle costs, labor and repair time after a head-on impact: sacrificial, reusable, and low maintenance.

17.8.3 Sacrificial

These units typically require full replacement or substantial repairs either on-site or in a maintenance yard following an impact. These units have lower installation 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.

17.8.3.1 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.

17.8.3.2 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 three (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**.

17.8.4 Location

Attenuators are ideally suited for use at locations where fixed objects cannot be moved, relocated, or made breakaway, and cannot be 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 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, TxDOT Roadway Standards, and the manufacturer's specifications for the specific crash cushion type.

17.8.5 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:

- Category;
- What is being shielded;
- Space available for installation;
- Debris produced by impact; and
- Ease and cost of maintenance.

17.9 Emergency Crossovers

Emergency crossovers between divided roadways are sometimes necessary for use by official vehicles (e.g., law enforcement, emergency responders, or traffic incident management). Coordination with local and State law enforcement and emergency services personnel is recommended to identify roadway sections where emergency crossovers may be necessary.

17.9.1 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 4-20**, use **TxDOT Roadway Standards - Barrier Gate** 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.

17.9.2 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.

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 as shown in **Figure 17-17**.

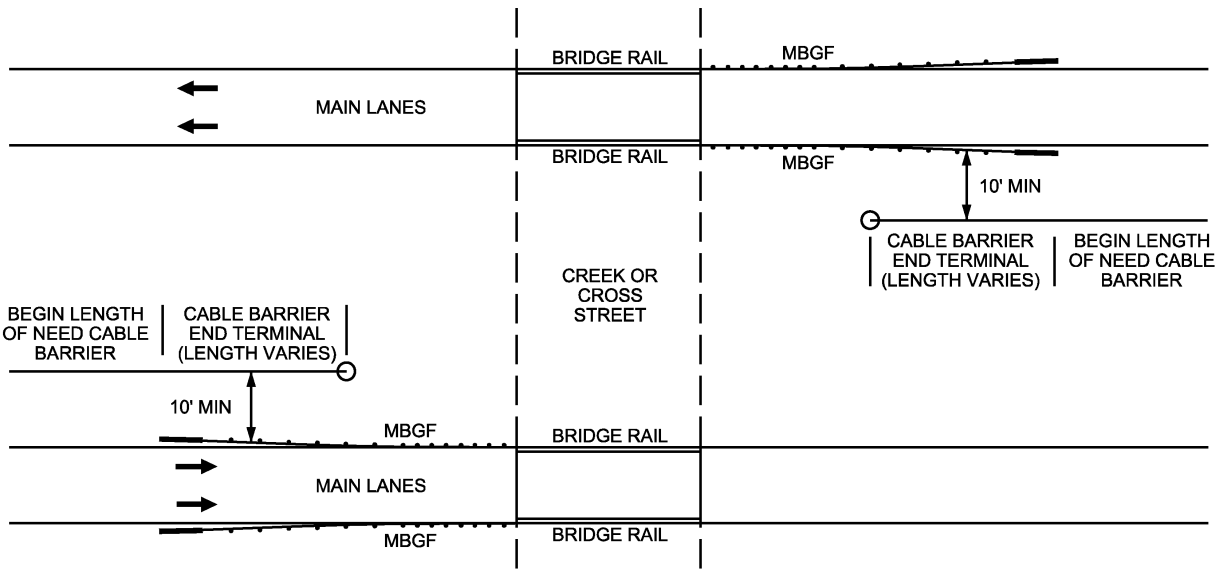


Figure 17-17: Cable Terminals Behind Metal Beam Guard Fence

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 **Figure 17-18**). 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.

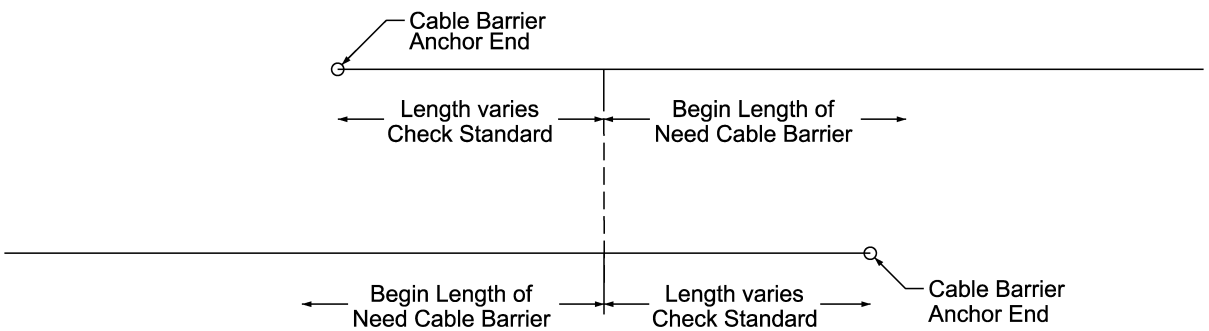


Figure 17-18: Recommended Cable Barrier Lap Length

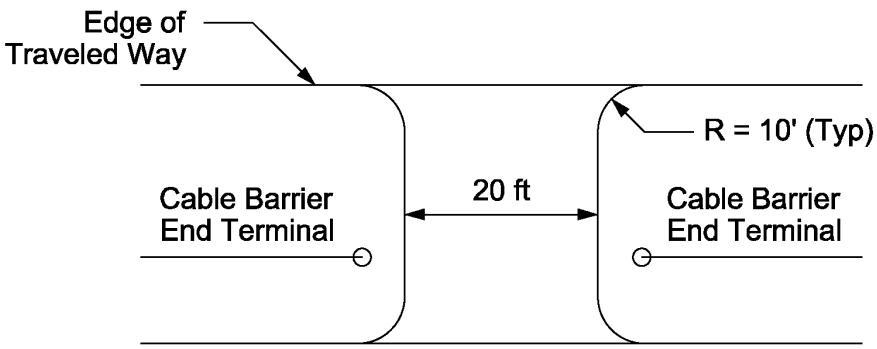


Figure 17-19: Another Typical Layout for Emergency Crossovers

Another typical layout for emergency crossovers is shown in **Figure 17-19**. 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.

Chapter 18
Bicycle Facilities

18.1 General

This chapter discusses bicycle facilities and includes the following sections and subsections:

18.1 General

- 18.1.1 Purpose;
- 18.1.2 Micromobility and Power-Driven Mobile Devices on Bikeways;
- 18.1.3 Definitions;
- 18.1.4 Relationship to Other Policies, Laws, and Regulations;
- 18.1.5 Projects That Can be Exempt from Bicycle Accommodations;
- 18.1.6 Use of Dimensional Values for Bikeway Design; and
- 18.1.7 Design Exceptions and Waivers for Dimensional Values of Bicycle Facilities.

18.2 Planning and Context

- 18.2.1 Bicycle Planning Principles;
- 18.2.2 Context Considerations;
- 18.2.3 Target Design User;
- 18.2.4 General Bikeway Selection; and
- 18.2.5 Bikeway Feasibility Assessment.

18.3 Elements of Design

- 18.3.1 Design Characteristics of Bicyclists;
- 18.3.2 Bicycles and Micromobility Device;
- 18.3.3 Bicycling Operating Space;
- 18.3.4 Bicyclist Operating Speeds;
- 18.3.5 Sight Distance;
- 18.3.6 Horizontal Shifting Tapers;
- 18.3.7 Surface Considerations;
- 18.3.8 Utility Considerations;
- 18.3.9 Drainage Considerations;
- 18.3.10 Bikeway Curb Considerations;
- 18.3.11 Railings and Barriers Adjacent to Bikeways;
- 18.3.12 Intersection Elements;
- 18.3.13 Bikeway Lighting; and
- 18.3.14 Restrict Motor Vehicle Use of Bicycle Facilities.

18.4 Bikeway Types

- 18.4.1 Shared Use Paths Adjacent to Roadways (Sidepaths);
- 18.4.2 Separated Bike Lanes;
- 18.4.3 Buffered Bike Lanes;
- 18.4.4 Bike Lanes;
- 18.4.5 Raised Bike Lanes;
- 18.4.6 Bike Accessible Shoulders;
- 18.4.7 Shared Lanes (wide outside lane); and
- 18.4.8 Rural Bikeway Types.

18.5 Intersections and Crossings

- 18.5.1 Principles of Intersection Design;
- 18.5.2 Intersection Approach Treatments;
- 18.5.3 Bicycle Ramp to Transition Between Bicycle Facilities;
- 18.5.4 Driveways;

- 18.5.5 Mid-Block Shared Use Path Crossings;
- 18.5.6 Roundabouts; and
- 18.5.7 Railroad Crossings.

18.6 Maintenance, Operations, and Work Zone

- 18.6.1 Maintenance Considerations; and
- 18.6.2 Temporary Traffic Control for Bicyclists/Maintenance of Traffic.

18.1.1 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 green-colored pavement markings, two-stage bicycle turn boxes, and intersection bicycle boxes are not currently prescribed in TxDOT policy or the 2011 TMUTCD. They will be incorporated into the next TMUTCD for use by cities and counties (not for TxDOT). However, it has not been released, so the 2011 TMUTCD remains effective until then. For the purposes of this guidance, in pictures, and figures where these devices are shown, they are for depiction purposes only to illustrate typical application of these treatments.

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 **TxDOT Traffic Standards**.

18.1.2 Micromobility and Power-Driven Mobility Devices on Bikeways

A variety of electrically powered micromobility and power-driven mobility devices have entered the market that may be operated on a street, sidewalk, or bikeway unless restricted by a jurisdiction. These devices are typically limited to top speeds of 30 mph or less. Research completed to-date shows that micromobility can replace many types of trips, including transit and/or rail, but it can also complement transit by providing first/last mile transportation.

The first modern bike share system in Texas was launched in 2011 by B-Cycle in San Antonio. The micromobility market continues to grow and diversify with the arrival of dockless bikeshare and e-scooters in 2017. E-scooter popularity has steadily increased since their introduction in 2017.

Like bicyclists, many of these users are uncomfortable operating on roadways in shared lanes with motor vehicle traffic, especially in areas with higher motor vehicle volumes and operating speeds. For purposes of this guidance, these users are assumed to be present and operating on bikeways. The **TTC §551.352** restricts E-Scooters operation on streets with a posted speed limit of 40 mph or higher.

When micromobility users do not responsibly operate their devices or properly park their devices, the micromobility devices can cause difficulties for other users of the transportation network – in particular pedestrians with disabilities. Individuals who are wheelchair

bound, have low-vision, or have other mobility issues can be significantly impacted by poorly parked shared micromobility devices. The provision of bikeways and adequate parking of these devices outside of pedestrian walkways can help ensure pedestrian facilities operate as intended. Some municipalities restrict micromobility device operation, parking, and/or operating speeds with geofence technology.

See **Section 19.2.2.** for further information on pedestrian accessibility requirements.

18.1.3 Definitions

The following definitions are provided for the purpose of this Guide; therefore, definitions may vary when reviewing other sources.

- **Bicyclists** - Able-bodied adult bicyclists, children, disabled bicyclists using adaptive/assistive equipment, recumbent bicyclists, bicyclists pulling a trailer as well as other micromobility users who operate on bikeways and use bicycle facilities.
- **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 (SUPs), and separated bike lanes. A bikeway does not include shared lanes (with or without shared lane markings), sidewalks, and signed bicycle routes.
- **Bicycle Facilities** - A general term denoting provisions to accommodate or encourage bicycling or travel with micromobility devices, including bikeways, bicycle detection, shared lanes with shared lane markings, wayfinding for signed bicycle routes, as well as parking and storage facilities.
- **Micromobility** - Small, low-speed (devices designed to travel at or below 30 miles per hour); human- or electric-powered transportation device, including bicycles, scooters, electric-assist bicycles, electric scooters (e-scooters), and other small, lightweight, wheeled conveyances.
- **Motor Assisted Scooter** - A self-propelled device with: i) at least two wheels in contact with the ground during operation; (ii) a braking system capable of stopping the device under typical operating conditions; (iii) a gas or electric motor not exceeding 40 cubic centimeters; (iv) a deck designed to allow a person to stand or sit while operating the device; and (v) the ability to be propelled by human power alone (**TTC §551.351**).
- **Power-Driven Mobility Device** - Any mobility device powered by batteries, fuel, or other engines – that is used by individuals with mobility disabilities for the purpose of locomotion, including golf cars, electronic personal assistance mobility device, such as the Segway® PT, or any mobility device designed to operate in areas without defined pedestrian routes, but that is not a wheelchair.
- **Shared Micromobility** - A device part of a network of shared services which may

requiring using vehicle located in fixed parking stations (docking stations) or may be available as "dockless" devices.

18.1.4 Relationship to Other Policies, Laws, and Regulations

18.1.4.1 AASHTO and FHWA Guidelines

The bicycle accommodation design guidance in this chapter 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 **AASHTO Bike Guide** continues to be the governing document for specific design criteria that is not contained within this chapter. The **FHWA Bikeway Selection Guide** may also be considered as resource, and in instance of contradictions, the **AASHTO Bike Guide** shall take precedence over the **FHWA Bikeway Selection Guide**. For further information on FHWA's position on design flexibility, refer to the August 2013 memo "**Bicycle and Pedestrian Facility Design Flexibility**."

18.1.4.2 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.

18.1.4.3 National Statutes

Under **23 USC 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."

18.1.4.4 Texas Codes

The TAC and the TTC provide directives for the design of bicycle facilities in the State of Texas.

TTC §551.002 defines micromobility devices and how they may be operated in Texas. On the road, bicyclists and micromobility riders must obey the same traffic laws, traffic signals, and street signs as motorists; however, e-scooters have specific restrictions on which roadways they can operate. E-scooters and bicycles are not considered motor vehicles. Laws and provisions related to bicycle operation apply to moped and e-bike operation.

TTC §201.902(c) requires TxDOT to adopt rules relating bicycle use on the roads in the state highway system. **43 TAC §25.53** specifies that TxDOT must take bicycle accommodation into consideration during the planning and implementation of all construction and rehabilitation projects. **43 TAC §25.54** 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 bicycle accommodation guidance is based on the review of national guidelines for best practices regarding 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 18.1.4.1** for document precedence.

Bicycle accommodations will be considered "provided" when bicyclists can navigate to logical termini throughout the project limits and have access to desired destinations. For example, a two-way shared use path (SUP) provided on one side of the travel way provides bicycle access along a roadway if there are no logical origins or destinations on the other side of the roadway (or none are anticipated during the lifetime of the project). For situations where a freeway or highway limits street connectivity, it may be necessary to provide bikeways on both sides of the freeway or highway to provide appropriate bicycle connectivity to the community.

18.1.5 Projects that can be Exempt from Bicycle Accommodations

Bikeways should be routinely included when planning and designing transportation facilities, addressing the needs of the target design user (see **Section 18.2.3**); consequently, all projects must be consistent with the identified needs for bikeways as identified in the environmental process.

Exemptions 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 exemptions 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 exemption based on the following criteria will be maintained with the project file with specific documentation as to the nature of the exemption, 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 18.1.7** of this manual. The **Statewide Planning Map** provides additional information on MPO boundaries and area types.

Projects located on the **Texas Bicycle Tourism Example Network** (found on the **Bicycle Tourism Trails Study webpage** on TxDOT.gov) are not exempt from providing bikeways specifically due to geographic location. Other exemptions (not specific to geographic location) may be used as appropriate.

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 18.2.4**, and are not exempt.

Off-system Bridges, with current ADT greater than 400 ADT, may be exempt from the bicycle clear space requirement when this addition may represent an unreasonable increase in cost. For specific Off-system bridge

requirements for a current ADT of 400 or less, see **Section 12.2**.

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 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 exemption, 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.
4. The project is in an urbanized setting with limited roadway improvements, such as 3R and 2R projects, 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.
5. 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 exemption should not be used if the project will help complete a gap in an overall bicycle network.
6. 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, funding opportunities for bicycle facilities can be found on **FHWA's Bicycle and Pedestrian Funding webpage**.
7. 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 preventive 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.

18.1.6 Use of Dimensional Values for Bikeway Design

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 **HCM**. Perceived safety is analogous to "subjective" safety as defined by the **HSM**.

While performance characteristics for micromobility devices are limited and not incorporated into this guidance, providing bikeway designs which accommodate adult bicyclists and the **Interested but Concerned Design User** (see **Section 18.2.3**) will generally lead to the establishment of geometric and operational design criteria that will result in bikeways that accommodate all users of bikeways including people using micromobility devices and personal assistive mobility devices.

The following terms are used throughout the guide to define the desirable, minimum, and constrained conditions for which bikeway widths will be determined:

18.1.6.1 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.

18.1.6.2 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.

18.1.6.3 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;
- At locations with low volumes of bicyclists where those volumes are anticipated to remain low; and
- Additional engineering countermeasures at locations should be considered where the use of constrained values is likely to increase crash risk or reduce bicyclist comfort.

18.1.7 Design Exceptions and Waivers for Dimensional Values of Bicycle Facilities

As previously discussed, the process for deciding if projects can be exempt 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 18.2.5**. A design exception or design waiver is needed if the minimum criteria for the selected facility is not met.

18.1.7.1 Urbanized Context (Urban Core/Urban/Suburban/Rural Town)

18.1.7.1.1 Design Exceptions

Bike Lane: If the minimum width specified in **Appendix A** is not met.

Shared Lane (Wide Outside Lane): If the traffic volume, speed, or width criteria (14-ft maximum, 13-ft minimum) specified in **Appendix A** are not met.

18.1.7.1.2 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 **Appendix B**, and the associated **AASHTO Bike Design** criteria are not met.

Separated Bike Lane/Buffered Bike Lane: If the minimum criteria specified in **Appendix B** are not met.

18.1.7.2 Rural Context

18.1.7.2.1 Design Exceptions

Shared Lane (Wide Outside Lane): If the traffic volume, speed, or width criteria (14-ft maximum, 13-ft minimum) specified in **Appendix A** are not met.

18.1.7.2.2 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 **Appendix B**, and the associated **AASHTO Bike Design** criteria are not met. For an unconstrained condition, a design waiver is required if width is between 8 to 10-ft. Any width below 8-ft will not be considered a bicycle accommodation or two-way shared use path operation. Widths between 6 ft and 8 ft would require one-way operation for bicyclists and may result in increased conflicts between bicyclists and pedestrians.

Bike Accessible Shoulder: For new construction, reconstruction, or widening projects in a rural setting where ROW is being acquired, a Design Waiver is required if a

minimum width defined in **Appendix B** for each bikeway type is not provided unless a SUP is being provided for bike accommodations (see **Section 18.4**).

18.2 Planning and Context

18.2.1 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 18-1**.

Table 18-1: Bicycle Planning Principles

Principle	Notes
Safety	• Physically 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. • Minimize or avoid conflicts with pedestrian traffic. • Provide sufficient shy distance to obstacles. • Establish geometric criteria that provide a comfortable facility to operate a bicycle including: ○ smooth vertical transitions with stable and slip resistant surfaces (Section 18.3.7), ○ accommodating side-by-side riding and passing at higher volume locations through provision of wider bikeways.
Connectivity	• Accommodate local bicycle and transit transportation routes and networks. • Connect bikeways and intersecting streets at a local scale for access to destinations, and at a regional scale for mobility. • 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 and other bikeway users, especially at conflict points. • Provide clear and intuitive transitions between different yet connected bicycle facility types. • Extend bikeways to logical and safe termini.

18.2.2 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 **Section 3.3**);
- **Intended Bicycle Accommodation Users** - Target design user (interested but concerned or all ages and abilities);
- **Approved Bicycle or Transportation Plan** – **Texas Active Transportation Plan Inventory**; and
- **Other Roadway Users** - Truck percentage and key movements, transit operation (headway) and key stops, curbside lane activity, expected pedestrian demand.

18.2.2.1 Land Use

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. For the purposes of bikeway selection, the five context classifications listed in **Section 3.3** have been consolidated into the following two groupings:

- Urbanized which includes the Urban Core, Urban, Suburban and Rural Town Contexts; and
- Rural Contexts.

18.2.2.2 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 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. Refer to

Bicycle and Pedestrian Count Program website for additional resources and information on bicycle and pedestrian count guidance. Note that the anticipated growth in usage should also be considered when defining the footprint for the bicycle accommodations.

18.2.2.3 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 18-2**.

Table 18-2: 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 (>5%) 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.
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, high ride-hailing demand, 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.

Factor	Description and Design Considerations
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 reduced traffic awareness skills. They can also create more conflicts with pedestrians when they are expected to share the same space.
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 ROW, 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.

Factor	Description and Design Considerations
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 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.

18.2.3 Target Design User

Different bicycle riders may have varying tolerances associated with the importance of the individual planning principles outlined above. **Figure 18-1** indicates the array of potential bicycle riders that should be considered when scoping and designing a roadway project.

Additionally, as discussed in **Section 18.1.2**, micromobility users must share facilities with other users of the transportation system and should be expected to operate within bikeways where provided. Where bike lanes are not provided, these users will operate within shared travel lanes or sidewalks which can raise safety concerns for users who might be apprehensive about walking, riding, or driving alongside these devices. Some micromobility companies advise their users not to ride on the sidewalks which relegates their users to using bikeways or sharing motor vehicle lanes. To mitigate these safety concerns bicycle networks can be expanded to also serve micromobility use.

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. Accommodating these users will also accommodate a wide spectrum of micromobility users.



Comfort Typology of Bicyclists

Design User Profile	Non-Bicyclist	Interested but Concerned	Somewhat Confident	Highly Confident
Bicycling Preferences	Uncomfortable bicycling in any condition, have no interest in bicycling, or are physically unable to bicycle.	Often not comfortable with bike lanes, may bike on sidewalks even if bike lanes are provided; prefer off-street or separate bicycle facilities or quiet or traffic-calmed residential roads. May not bike at all if bicycle facilities do not meet needs for perceived comfort.	Generally prefer more separated facilities, but are comfortable riding in bicycle lanes or on paved shoulders if need be.	Comfortable riding with traffic, will use roads without bike lanes.
Percent of Population	31-37%	51-56%	5-9%	4-7%

Figure 18-1: Types of Bicycle Facility Users as a Percentage of Total General Population

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. 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.

An exemption to this bikeway selection approach are situations where bicyclists are operating in large groups, or higher speed bicyclists are likely operating on a sidepath. In these situations, it is acceptable to provide multiple facility types (e.g., a wide outside lane and sidepath, or a bicycle lane and sidepath) where practicable. In all locations with bikeways, there still may remain individual bicyclist who prefer to operate at higher speeds

(typically greater than 20 mph) who may prefer to operate in the travel lane.

18.2.4 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 18.4** 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 18.4** 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 18.4** for a specific bikeway, **the engineer has the discretion to select the bikeway that is best suited to the project needs and constraints**.

Figure 18-2 provides an example selection process from FHWA. Note that **Figure 18-3** and **Figure 18-4** should be used to make the initial recommendation for the type of bicycle facility.

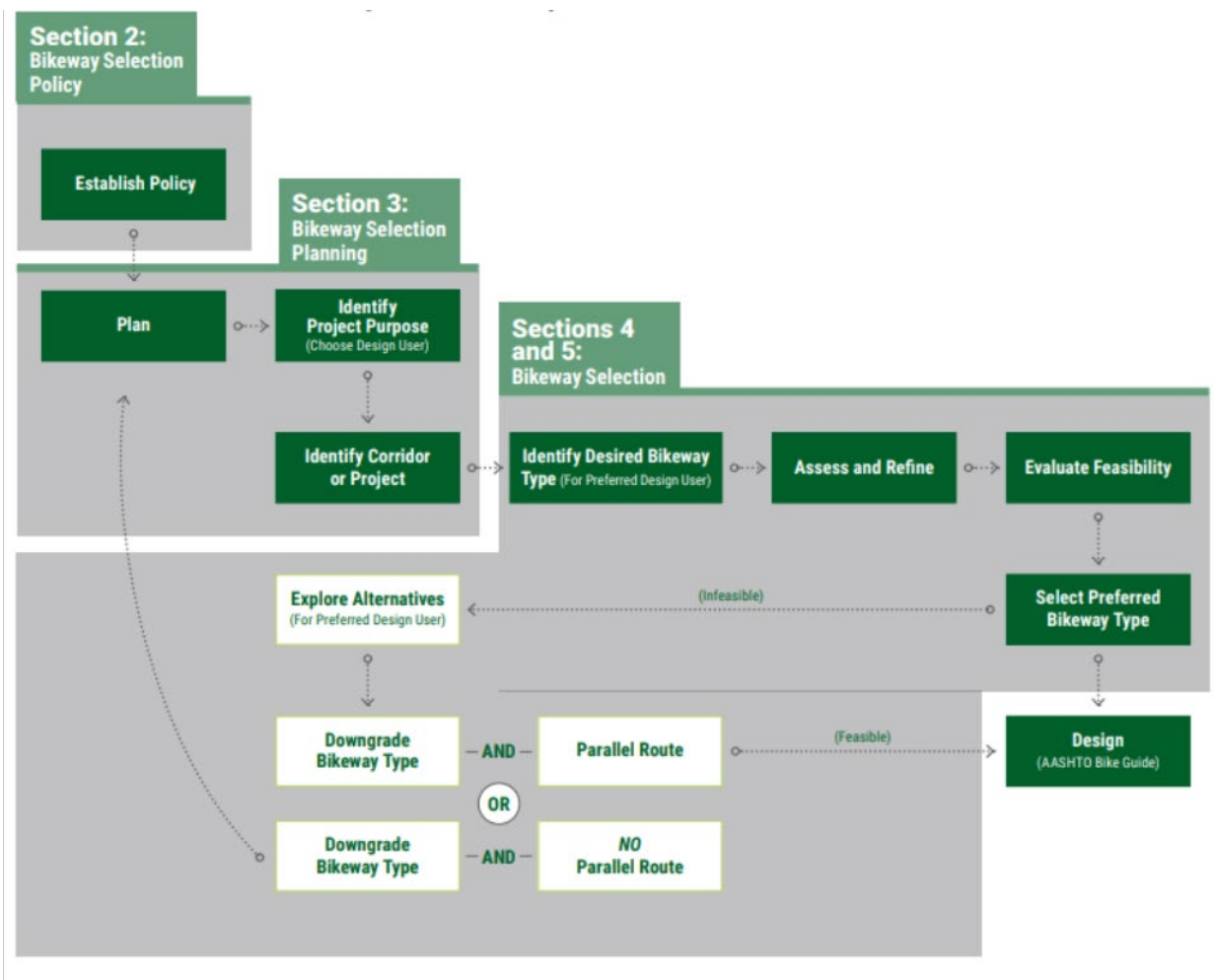


Figure 18-2: FHWA Bikeway Selection Process and Guide Outline

18.2.4.1 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 ROW or with additional ROW, the project should use **Figure 18-3** for the initial recommendation for bikeway facility type. See **Section 18.2.5** 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 Bike Guide** and, for separated bike lanes, **FHWA Separated Bike Lane Planning and Design Guide** and **FHWA's Separated Bike Lanes on Higher Speed Roadways: A Toolkit and Guide**. Note: TAS/ **ADA Accessibility Guidelines (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 **Section 12.2** for specific off-system bridge requirements for current ADT of 400 or less. Where feasible, desirable shoulder width as shown in **Figure 18-4** should be used. (This requirement is not applicable if an SUP or other bikeway is provided for bike accommodations.)
- For roadways identified on the **Texas Bicycle Tourism Example Network** , preferred 10-ft (minimum 8-ft) shoulder width should be provided on bridges. (This requirement is not applicable if an SUP or other bikeway is provided for bike accommodations.)

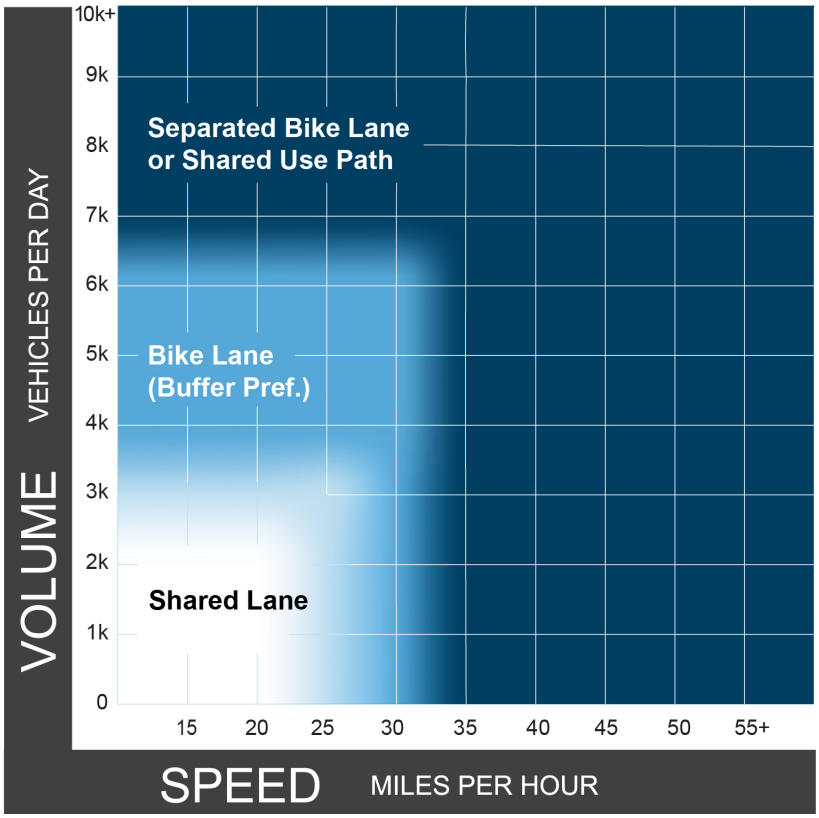


Figure 18-3: Recommended Bicycle Facility Selection for Urban, Urban Core, Suburban, and Rural Town Context

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.

18.2.4.2 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 ROW is being acquired, the following guidance is provided:
    - When the scoping process and environmental documentation indicates a need for bicyclist accommodations, the recommended bikeway is shown in **Figure 18-4**, indicating the desirable shoulder widths for various speeds and traffic volumes. In some cases, a shared use path or other locally preferred facility type (such as a buffered bicycle lane or separated bicycle lane) may be identified during stakeholder outreach. See **Section 18.4.6** for

additional guidance on shoulders and **Section 18.4.7** for shared lanes. See **Section 18.2.5** 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 ROW, the above should be followed where feasible. See **Section 18.2.5** 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 **Section 12.2** for specific off-system bridge requirements for current ADT of 400 or less. Where feasible, desirable shoulder width as shown in **Figure 18-4** should be used. (This requirement is not applicable if an SUP or other bikeway is provided for bike accommodations.)
- For roadways identified on the **Texas Bicycle Tourism Trails Example Network**, preferred 10-ft (minimum 8-ft) shoulder width should be provided on bridges. (This requirement is not applicable if an SUP or other bikeway is provided for bike accommodations.)

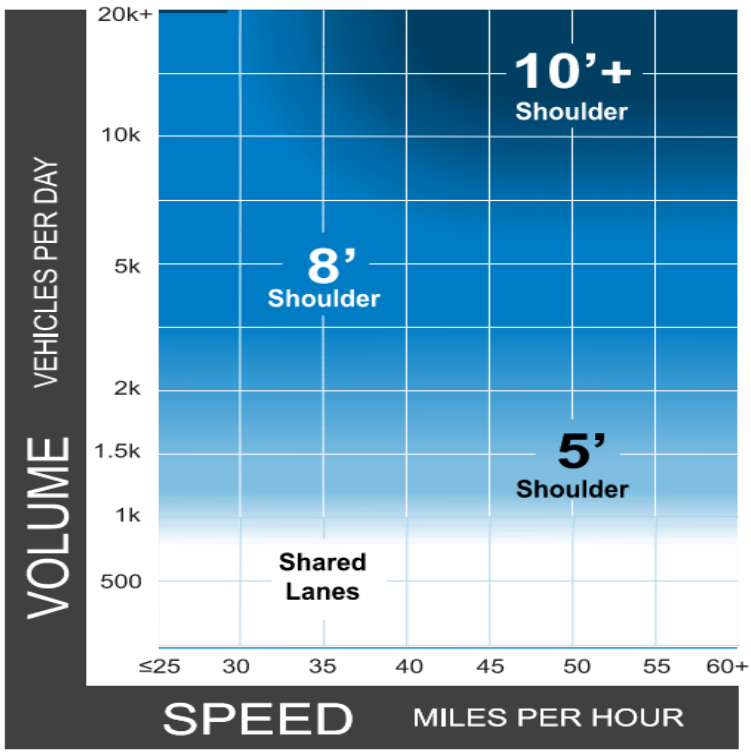


Figure 18-4: Recommended Bicycle Facility Selection for Rural Context

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.

18.2.5 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.

18.2.5.1 Prioritizing Safety

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 18.1.6**, 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 bikeway approaches a large intersection, several decisions have to be made that can impact safety. In locations with constrained ROW, 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 18.5.2** 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. See **Section 18.5.2.6** for information on protected intersections.

18.2.5.2 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 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.

18.3 Elements of Design

18.3.1 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 exemptions 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.

18.3.2 Bicycles and Micromobility Devices

Some of the types of devices that are commonly used on Texas streets and trails are shown below in **Figure 18-5**. 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. See **Chapter 22** for additional transit implications.

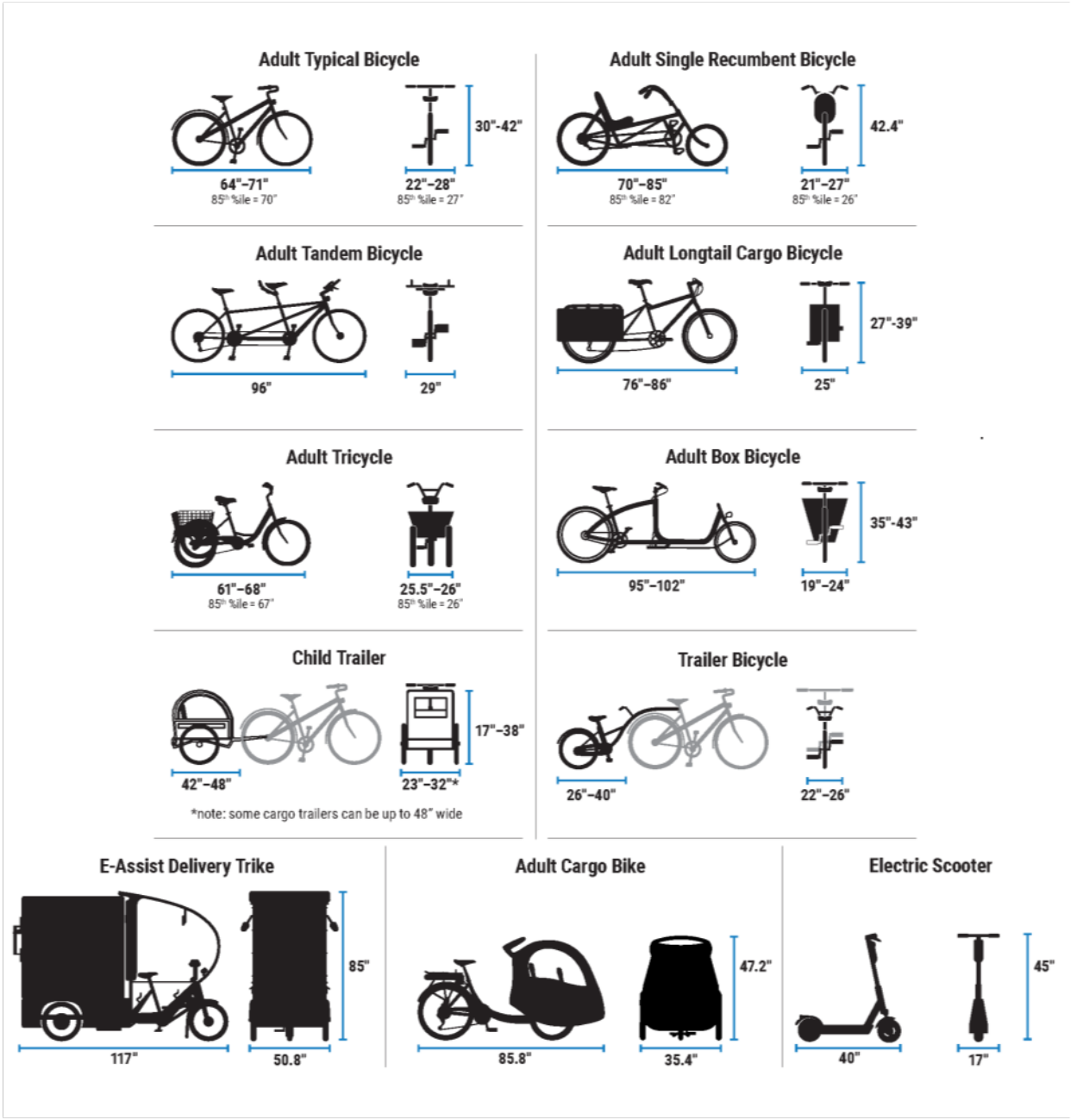


Figure 18-5: Typical Dimensions of Bicycle and Micromobility Types

18.3.3 Bicycling Operating Space

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-in horizontal width;
- 10-ft length; and
- 7-ft height.

Figure 18-6 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 as well as other Micromobility device users. This lane does not need to be marked on a bikeway unless it is operationally beneficial to do so. The bikeway width minimums provided in this guide account for the operating and shy spaces required for an adult bicyclist operating with a trailer.

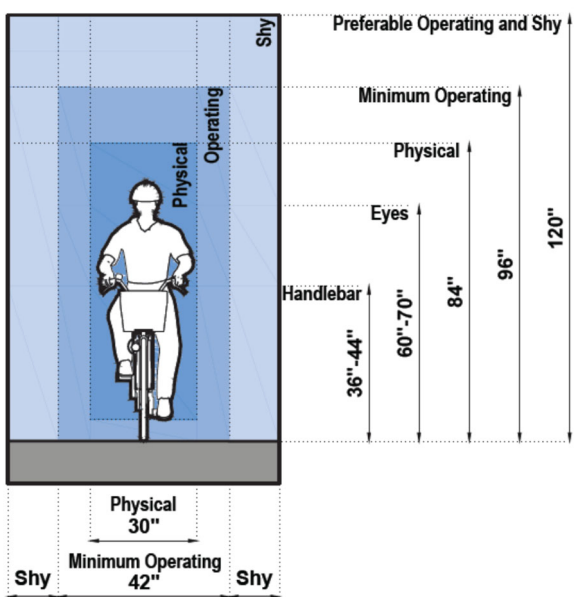


Figure 18-6: Operating Space for Bicyclists

18.3.4 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 18.4 as appropriate.

18.3.5 Sight Distance

Section 4.11 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.

18.3.5.1 Bicycle Stopping Sight Distance

Adequate motor vehicle stopping sight distance is important for the safety of pedestrians and bicyclists who must cross roadways. See Section 4.11.4 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. Stopping sight distance can also be calculated using the formula shown in **Figure 18-7**. The table in **Figure 18-7** shows the minimum stopping sight distance for a typical adult bicyclist under wet conditions at various design speeds and grades based on a total perception and brake reaction time of 2.5 seconds.

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: ± = 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 18-7: Stopping Sight Distance Formula and Table

18.3.5.2 Intersection Sight Distance

Section 4.11.4 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 Section 4.11.4 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. Sight triangles should remain clear between 2.3-ft and 7-ft measured

above the ground surface to account for the wide range of bicyclist eye heights and recumbent bicyclists as well as to ensure the visibility of child bicyclists to approaching motorists.

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.

18.3.5.3 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.

18.3.5.3.1 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 18-8**. 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.

18.3.5.3.2 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 18-8**. 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; and

- **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 18-3** 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. The values for clear space in **Table 18-3** are measured back from the point of curvature of the motorist's effective turning radius (i.e., fastest path), 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.

Table 18-3: 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

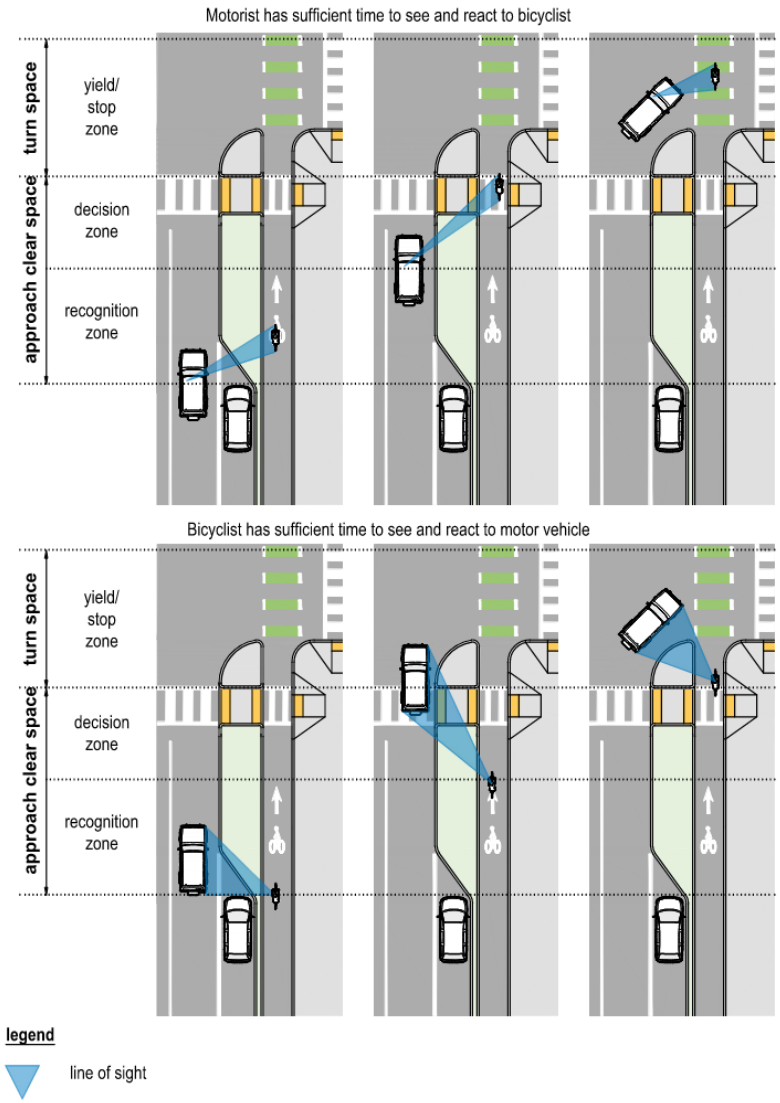


Figure 18-8: Yielding Scenarios Case A

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.3.5.3.3 Case A: Right-Turning Motorist Across Separated Bike Lane or Sidepath

Figure 18-8 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 18-3** 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.

18.3.5.3.4 Case B: Left-Turning Motorist Across Separated Bike Lane or Sidepath

Figure 18-9 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 18-3 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 conditions present:

- High traffic volumes and multiple approach lanes;
- Higher operating speeds; and
- High left turn volumes.
- High volumes of right-turn-on-red motorists from the intersecting side street, where turning motorists:
    - may not be scanning for bicyclists or pedestrians approaching from the right; or
    - block the crosswalk requiring bicyclists and pedestrians to cross behind them limiting their visibility to left-turning motorists from the main roadway.

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, to 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 18.5.2).

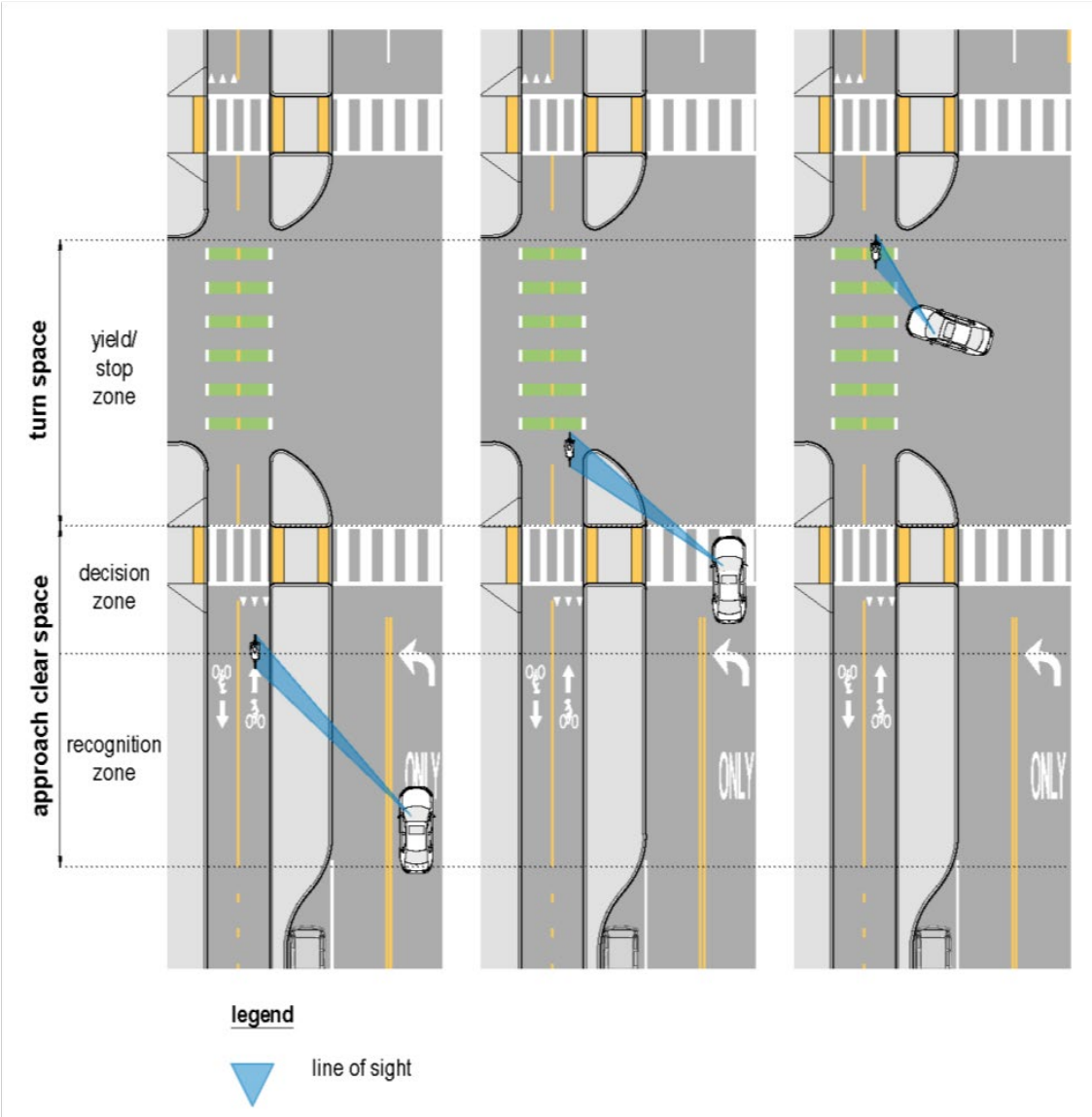


Figure 18-9: Intersection Sight Distance Case B

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.3.6 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 18-4**.

Table 18-4: 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 igure C-8. See guidance on design speed for each bikeway type throughout **Section 18.4**.

18.3.7 Surface Considerations

18.3.7.1 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. Past research has shown that smoother surfaces are desirable for bicyclists. When placing a seal coat as a final driving surface, consider one of the following low cost options to provide an improved riding surface for bicycles: use a smaller seal coat aggregate, use a smaller seal coat aggregate on the shoulders, use fog seal on existing seal coated shoulders rather than a new seal coat, or use smaller aggregate for the top course of multiple course seal coats.

18.3.7.2 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.

18.3.7.3 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.

18.3.8 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.

18.3.9 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; and
- 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 18.3.6** and refer to the **TMUTCD**); and
    - 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 18-10**);
        - Limit the gap between the grate and frame to 0.5 inch or less, perpendicular to the path of travel; and
        - Grates should remain smooth and flush with the bikeways surface.

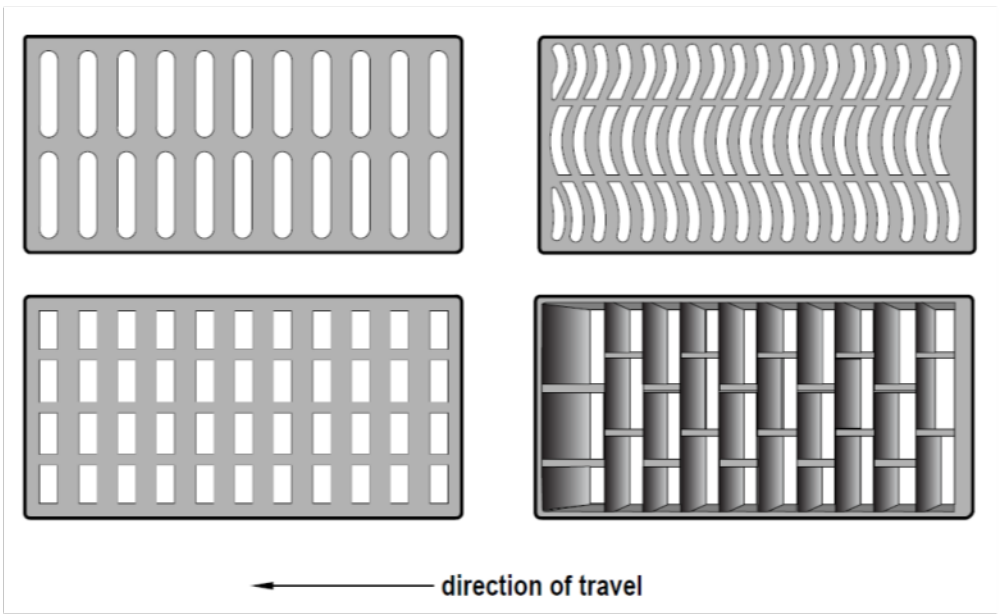


Figure 18-10 : Bike Compatible Drainage Grates

18.3.10 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-in in height there is no minimum shy space; however, shy space to other appurtenances should be carefully considered. For curbs between 3-in and less than 6-in in height, a desirable shy space of 1-ft, with no minimum in constrained conditions, applies. Refer to the current **TxDOT Roadway Standards = 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.

18.3.11 Railings and Barriers Adjacent to Bikeways

A barrier that separates vehicular traffic from a sidepath or separated bike lane must:

- Meet the current barrier standards for the vehicular speed; 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 required. Barriers across bridge structures must meet the respective TxDOT's **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-in 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 shared use path (SUP) or other bikeways. (see **Figure 18-11**).

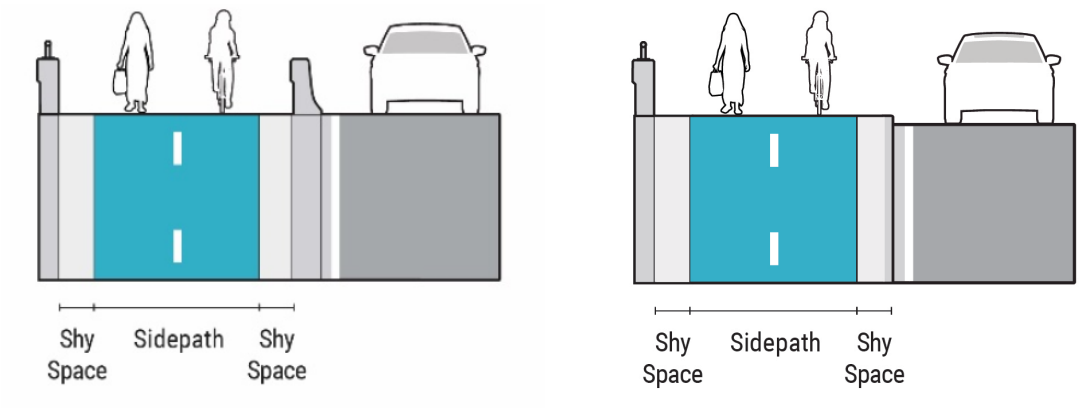


Figure 18-11: Example Bridge with Barriers for Sidepaths; High-speed (left) and Low-speed (right)

Where shared use sidepaths or separated bike lanes with sidewalks 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 sidepath width from desired to constrained minimum;
- Narrow separated bike lane width from desired to minimum; or
- Narrow sidewalk width to the minimum.

18.3.12 Intersection Elements

18.3.12.1 Bicycle Crossing / Conflict Markings

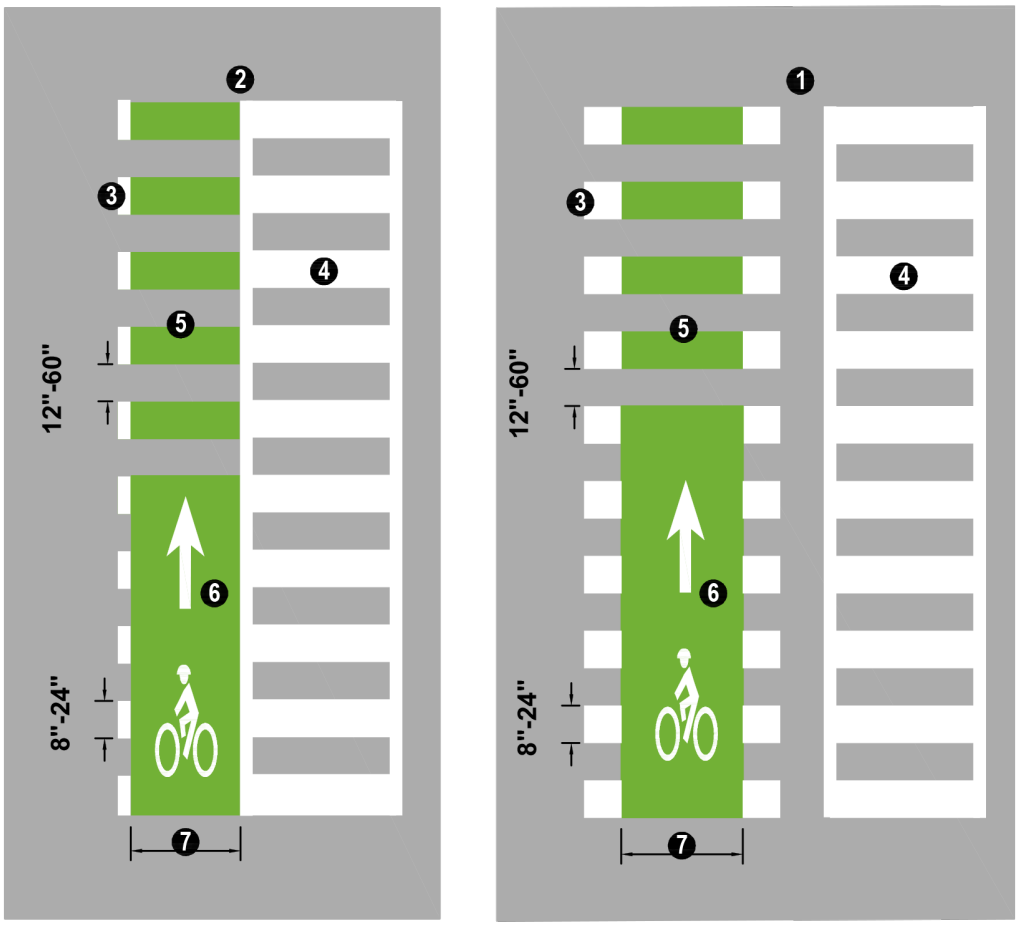
Where a bike lane crosses an intersection separate from a crosswalk, bike lane markings may be extended through the intersection to delineate the bicycle crossing. Bike lane crossings are desirable to:

- Delineate a preferred path for people bicycling through the intersection, especially a crossing of a wide or complex intersection;
- Improve the visibility and recognizability 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 18-12 provides design details for various bike crossing scenarios.

Where marking the conflict area is desirable, a dotted white edge line should be used to delineate the bicycle lane extension through the intersection by marking the left and right edge of the bicycle lane. The dotted lines are 2-ft in length with 2-ft to 6-ft gaps located on the

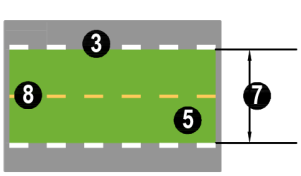
edge of the bike lane. Normal width (6-in) or wide lane line widths (8-in) may be used, but they should typically match the width of the bike lane lines. Conflict area visibility may be enhanced with green-colored pavement and a bicycle lane symbol. If used, green-colored pavement should generally match the pattern of the dotted edge lines but may be solid where additional emphasis of the crossing is desired (see **Section 18.1.1** for conditions of allowed use of green-colored pavement marking).



Detail 1
one-way bicycle crossing
with no offset from
marked crosswalk

Detail 2
one-way bicycle crossing
with 1' min offset from
marked crosswalk

1. 1' min offset
2. no offset
3. normal or wide white dashed line
4. pedestrian crosswalk
5. optional green dotted or solid colored pavement
6. optional bicycle symbol
7. match width of bike lane
8. 4"-6" yellow centerline



Detail 3
two-way bicycle
crossing

Figure 18-12: Bicycle Crossing Pavement Markings

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.3.12.2 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" (Signs R10-40 or R10-41) sign installed immediately adjacent to the traffic 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 18-13**;
- 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 R10-40 or R10-41 sign(s) 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. Refer to the **TMUTCD** for additional guidance on the use of Bicycle Signal Faces.

18.3.12.2.1 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 18-13** 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.

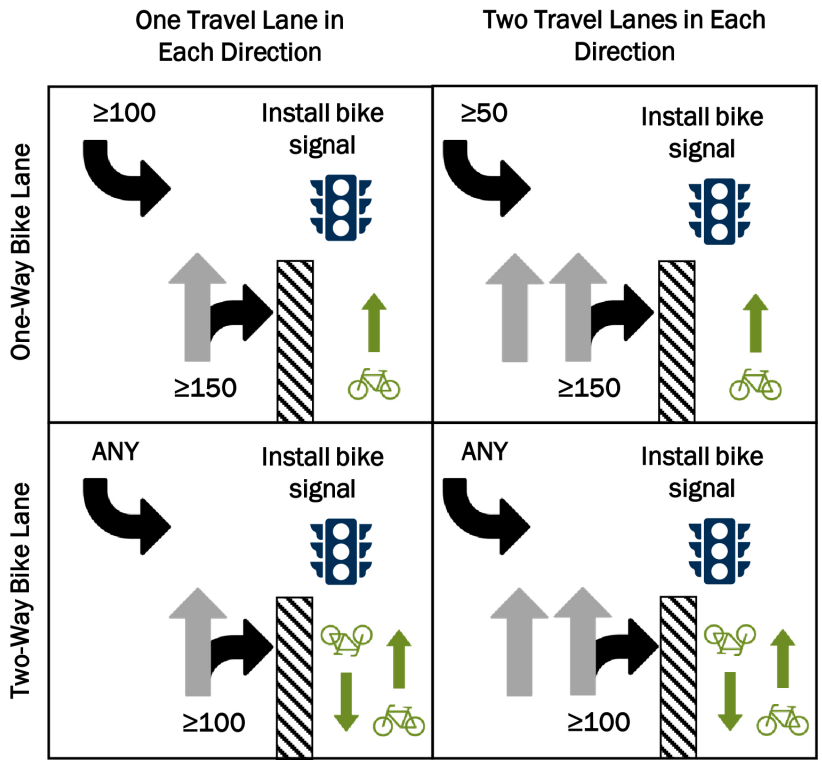


Figure 18-13: Signal Phase Separation of Turning Motorists

18.3.13 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 that 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 intersections where bikeways cross;
- At trail intersections with higher volume roadways; and
- At major trail entrances.

Designers should see **Chapter 19** for information and guidance on the design of pedestrian scale lighting and appropriate illumination levels. Bikeway lighting should follow the same principles.

18.3.14 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 ROW that may appear to look like roadway or driveway entry points to motorists.

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 or anticipated high likelihood 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 injury. Approaching bicyclists may shield these obstructions 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.

A strategy to minimize entry of motorists is to design the shared use path entry point as two sections separated by a conspicuous center island which may include low landscaping. The trail approach to the split should be delineated with solid line pavement markings to guide the path user around the island. Each half section of the path should be no more than 6 ft wide to discourage entry by motorized vehicles while still serving path users.

The center island and landscaping (if provided) should be designed to allow emergency and maintenance vehicles to enter the shared use path, if needed, by straddling the island and passing over the island and/or landscaping. Alternatively, it may be more appropriate to designate emergency and maintenance vehicle access via separate access drives adjacent to trail access points, which can be secured by gates or fencing. Such separate access drives should not be secured only by removeable bollards, as this may encourage potential use by trail users. Where a device to restrict motorists is determined to be necessary, consider the use of flexible or spring-mounted delineators before installing rigid bollards to reduce risk of potential injury to shared use path users. At locations where rigid bollards within the bikeway are determined to be necessary, the following should be provided:

- Bollards which are visible to approaching users meeting TxDOT specifications. Bollard placement should allow sufficient width for safe passage of bicyclists and other users by ensuring a minimum width of 6 ft is provided between bollard posts or a bollard and

another vertical obstruction or edge of paved surface;

- Retroreflective pavement markings following **TMUTCD** guidance for obstructions on bikeways should be provided to guide users around the bollard; and
- For bollard retroreflectivity, and color marking guidance, refer to the **TMUTCD**, and the applicable **TxDOT Special Specification 5033 - Bollards**.

18.4 Bikeway Types

This section provides a description and brief design guidance for the most common bicycle facility types. From left to right, **Figure 18-14** 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 18.2.3**) since they have the most separation from motor vehicles.

A detailed description of each bikeway for urbanized contexts can be found starting in **Section 18.4.1** through **Section 18.4.7**. Rural contexts can be found in **Section 18.4.8**. For additional guidance not stated herein, see additional design resources listed in **Section 18.6.2**, 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 the **Texas Bicycle Tourism Trails Example Network**, provide appropriate Bike facility in coordination with the local community.

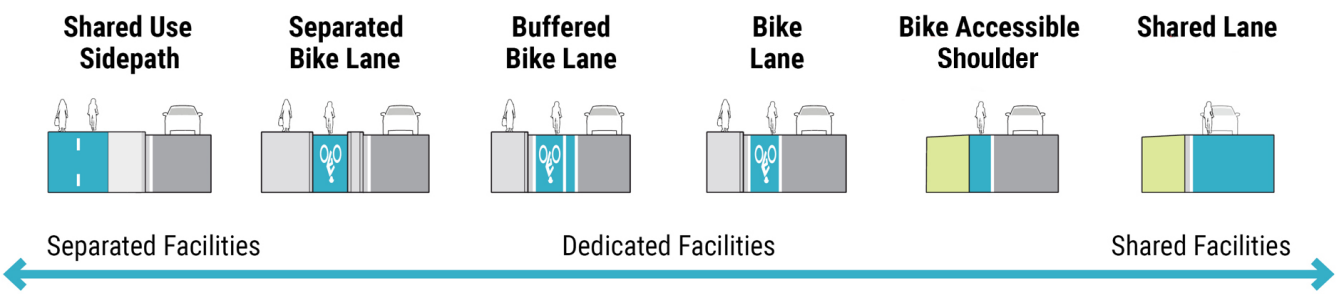


Figure 18-14: Bicycle facility Types by Level of Separation

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.

18.4.1 Shared Use Paths Adjacent to Roadways (Sidepaths)

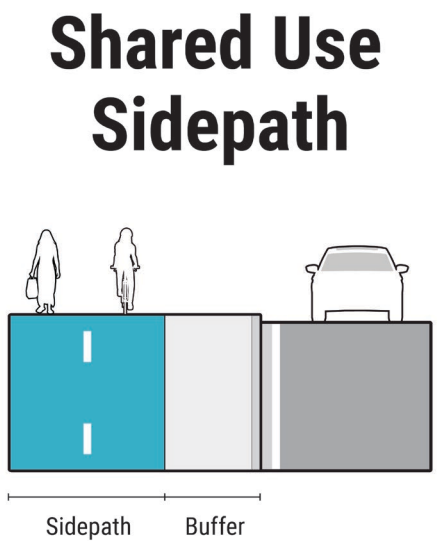


Figure 18-15: Example Shared Use Path Section

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 are likely to include inline skaters, skateboarders, pedestrians, and other micromobility or personal mobility device users. 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 18.5**). 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.

While the following guidance is written from the perspective of sidepaths located within roadway corridors, it is also applicable to shared use paths in independent alignments. Shared use paths in independent alignments often have fewer conflict points than sidepaths, however they may require the provision of more access points to provide connectivity to desired origins and destinations. The most common shared use path locations are through public property, parks, and abandoned railroad corridors. The **AASHTO Bike Guide** should be reviewed for additional guidance of shared use paths in independent alignments.

Refer to **Section 19.3.3** for additional guidance on Shared Use Paths.

18.4.1.1 Application

Sidepaths are particularly useful when roadway width is limited and providing an on-street bikeway is not feasible. However, before

defaulting to a sidepath, careful consideration should be given to potential conflicts between pedestrians and bicyclists as it is usually 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 Bike 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 that may tend to obstruct the sidepath usage or sight lines.

18.4.1.2 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.

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 18-5** 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 facility 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 (BP|CX)** contains pedestrian and bicyclist count data for various types of facilities statewide and can be used to supplement missing information.

The desired width for a sidepath 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 18.4.1**). 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 three primary strategies which can be considered for separating users on the facility (see **Figure 18-16)**:

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.

Alternatively, a separate bicycle facility type may be selected.

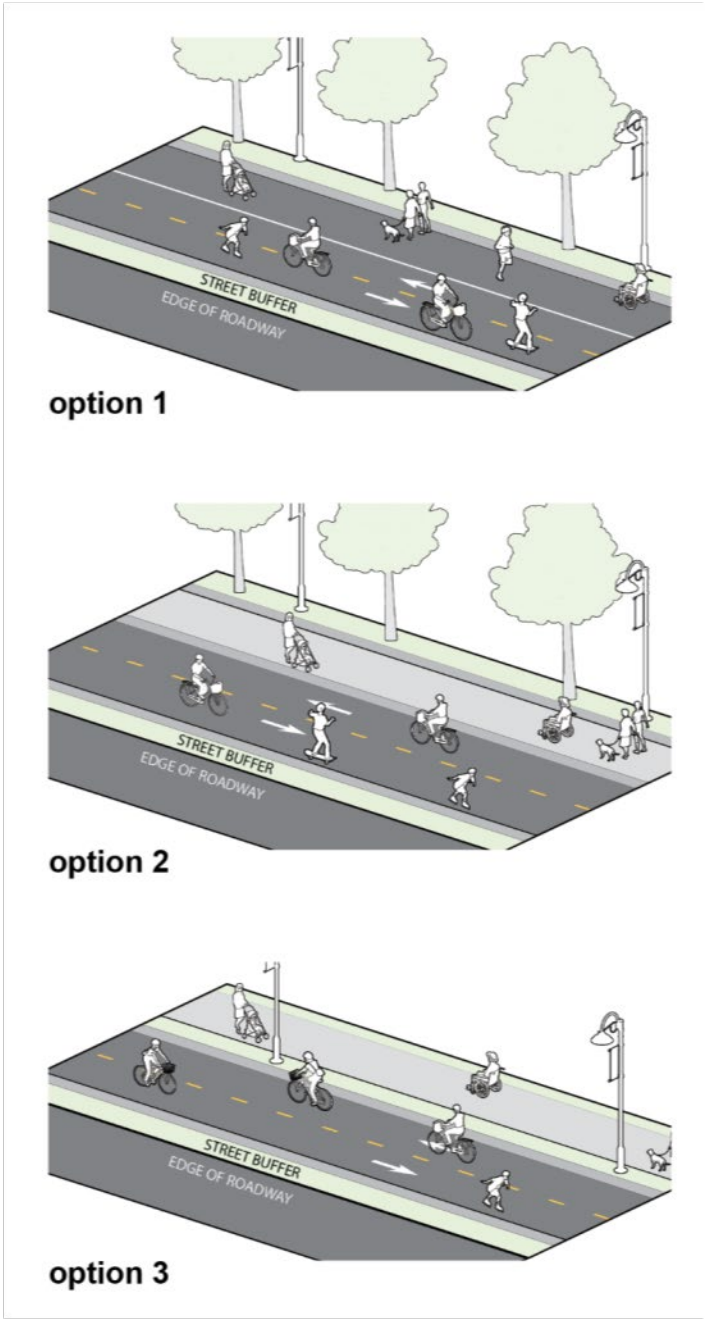


Figure 18-16: Potential Options for Separation of Shared Use Paths

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 18-5: Shared Use Path (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=Center-line	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

18.4.1.2.1 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 (if 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 18-6**.

Table 18-6: 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
Notes:	
1. For curb and gutter sections with a shoulder, bike lane or any buffer in addition to the curb offset the minimum street buffer measurement begins at the edge of through travel lane (through travel lanes do not include turn lanes, accel lanes, or decel lanes). 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 18-6.	

The *desirable street buffer* would be the respective desirable clear zone values from **Table 4-20**. 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, and utility poles) and continuous (e.g., walls, railings, fences, and 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, the 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, and 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 **Section 19.12**. Specific railing considerations for bicyclists are provided in **Section 18.3.11**.

18.4.1.2.2 Signing and Marking

Signs installed in the vicinity of sidepaths shall be installed in accordance with the applicable **TMUTCD** criteria.

The **TMUTCD** 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.

Refer to **TMUTCD** – Section 9C.03 Marking Patterns and Colors on Shared Use Paths for information about markings for sidepaths. Additional information is provided in **Section 18.5.5** regarding mid-block shared use path crossings.

18.4.1.2.3 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). 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.

18.4.1.2.4 Cross Slope and Grade

Cross slopes of 1 to 1.5% 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 2.0 percent.

For shared use paths on independent alignments, longitudinal grades must meet pedestrian accessibility guidelines as they are expected users of the facility. **PROWAG** provides minimum design guidance for pedestrian accessibility of shared use paths.

PROWAG reinforces that grades on shared use paths on an independent alignment are limited to 5 percent to maximize pedestrian accessibility. 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. Grades may also exceed 5 percent at overpasses and underpasses if ramps and landings complying with **PROWAG,** Section R407, are provided.

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 **Section 19.3**);
- 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 at each horizontal curve used to form the switchback;
- Provide resting intervals with flatter grades to permit users to stop periodically and rest; and
- Consider the provision of accessible pedestrian handrails.

Note: Grades steeper than 3 percent may not be practical for shared use paths with crushed stone or other unpaved surfaces for both bicycle handling and drainage erosion reasons.

When shared use paths follow the same alignment of the roadway, the sidepath grades will typically match the grade of that adjacent roadway. A sidepath within the roadway ROW of a road with a grade that exceeds 5 percent may not exceed the adjacent roadway grade.

18.4.1.3 Horizontal Geometric Design

Table 18-7 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 2.0 percent.

Table 18-7: Minimum Radii for Horizontal Curve on Paved Shared Use Path at a 20-Degree Lean Angle

Design Speed (mph)	Minimum Radius (ft)
8	12
10	18
12	27
14	36
15	41
16	47
18	60
20	74
25	115
30	166

18.4.1.4 Vertical Geometric Design

Table 18-8 provides the vertical curve length needed based on stopping sight distance and algebraic grade difference on crest vertical curves. **Table 18-9** provides the equation for calculating this length for scenarios not shown in **Table 18-8**. A recumbent bicyclist is recommended to establish design criteria and the following assumptions should be used:

1. The eye height of the recumbent bicyclists is 3.83-ft; and
2. 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 18-8: 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														

Table 18-9: Crest Vertical Curve Equation

Length of Crest Vertical Curve to Provide Sight Equations		
when $S > L$ $L = 2S - \frac{200(\sqrt{h_1}+\sqrt{h_2})^2}{A}$ when $S < L$ $L = \frac{AS^2}{100(\sqrt{2h_1}+\sqrt{2h_2})^2}$		
where:		
L	=	minimum length of vertical curve (ft)
A	=	algebraic grade difference (percent)

18.4.1.5 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 18.3.11 provides additional details on the design of barriers and railings. Designers should see **Chapter 19** 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.

18.4.2 Separated Bike Lanes

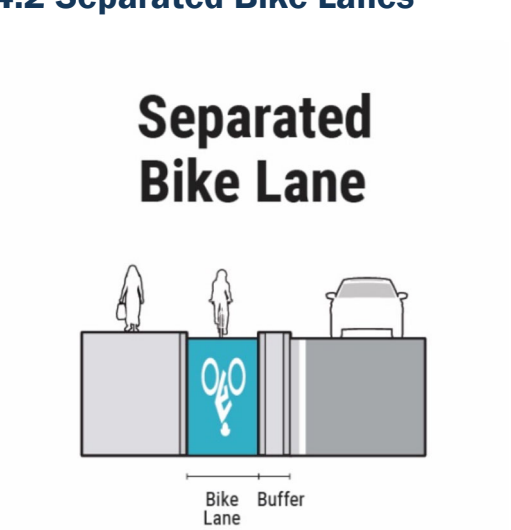
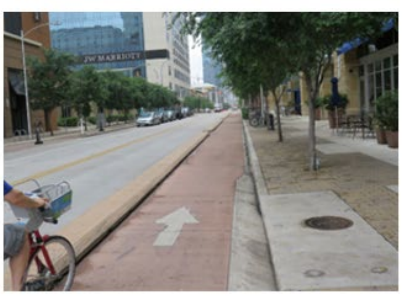


Figure 18-17: Example Separated Bike Lane Schematic

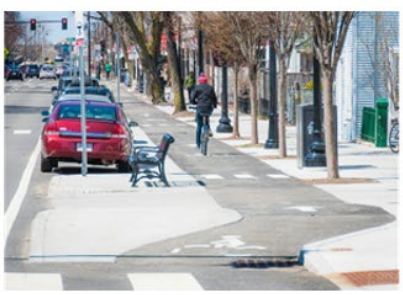
A separated bike lane is a bicycle lane that is physically separated from the adjacent motor vehicle traffic by vertical elements in the street buffer. They typically are designed to operate one-way but may also operate two-way. These are sometimes also referred to as protected bike lanes or cycle-tracks. 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 Separated Bike Lane Planning and Design Guide**, **FHWA's Separated Bike Lanes on Higher Speed Roadways: A Toolkit and Guide** and the **Massachusetts DOT Separated Bike Lane Planning & Design Guide**. See **Figure 18-18** for examples of separated bike lanes.



Austin, TX.
Street Level with Concrete Curb Separation



Cambridge, MA.
Sidewalk Level with Parking Separation



Raleigh, NC.
Street Level with Flexible Post Separation

Figure 18-18: Examples of Separated Bike Lanes by Separation Types

The cross section of a separated bike lane has three distinct zones (see **Figure 18-19**):

- **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.

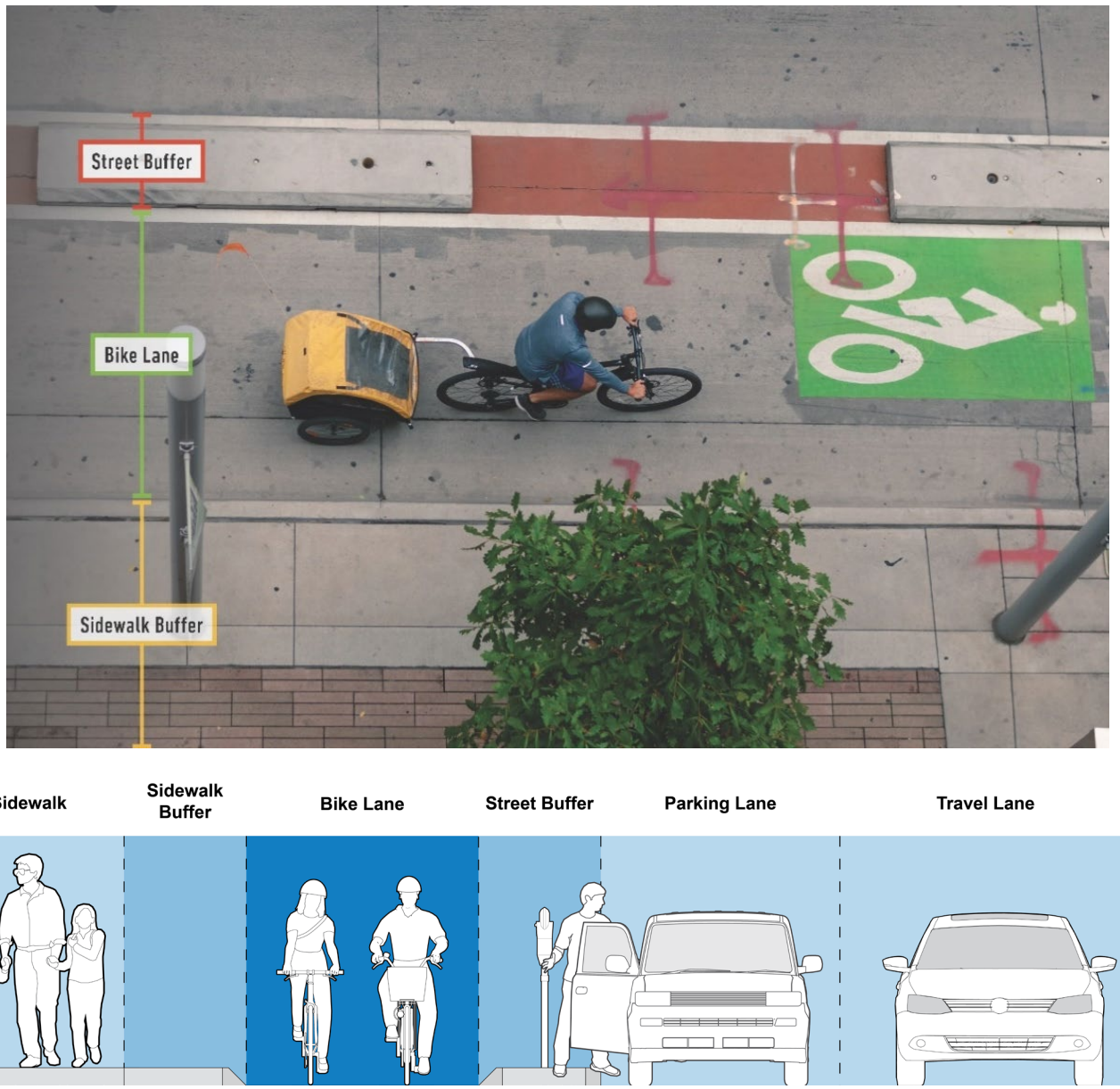


Figure 18-19: Separated Bikeway with Three Distinct Zones

18.4.2.1 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.

18.4.2.2 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 18-10** for one-way separated bike lanes. Two-way separated bike lane widths should follow the previous guidance in **Section 18.4.1** (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; and
- At locations with low volumes of bicyclists where those volumes are anticipated to remain low (< 50 bicyclists/hour).
- See **Section 18.4.2.6** for guidance on buffers on two-way separated bike lanes.

Table 18-10: One-Way Separated Bike Lane Widths (Recommended)

Peak Hour Directional Bicyclist Volume	One-Way Separated Bike Lane Width (ft) Recommended Values		
	Between Vertical Curbs without Gutter	Adjacent to One Vertical Curb	Between Sloped Curb, at Sidewalk Level, or Adjacent to Curb with Gutter
<150	6.5 - 8.5	6 - 8	5.5 - 7.5
150-750	8.5 - 10	8 - 9.5	7.5 - 9
>750	≥10	≥9.5	≥9
Practical Minimum*	4.5	4	4

*Peak Hour Directional Bicyclist Volume not applicable

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 **Figure 18-19** 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 18.4.2** for guidance on buffers on two-way separated bicycle lanes. Two-way separated bicycle lanes also need to meet the guidance in **Section 18.4.2** and **Section 18.5** with respect to intersecting driveways, and conflict points; and
- In scenarios where there is none, or limited space for a street buffer, designers may consider a raised bike lane, see **Section 18.4.5**.

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. Designers should ensure that the vertical elements are visible to approaching bicyclists and motorists. In some cases, vertical elements should be supplemented with a striped buffer to clearly delineate the buffer zone. The placement of vertical elements within the street buffer should also consider the need for shy distance to the bikeway and to the travel lane, and the door-swing of parked cars adjacent to areas of on-street parking. See **Section 18.4.3** for additional guidance for the marking of buffers.

18.4.2.3 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**.

18.4.2.4 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.

18.4.2.5 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.

18.4.2.6 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 ROW, curbside uses, intersection operations, ingress and egress at the termini, maintenance, and feasibility.

Table 18-11 and **Table 18-12** provide an overview of trade-offs for various scenarios.

Table 18-11: Separated Bike Lane Configurations on a One-Way Street

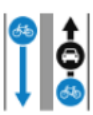
Corridor-level Planning Considerations	One-way SBL	Counterflow SBL	One-way SBL Plus Counterflow SBL	Two-way SBL
				
Access to Destinations	Limited access to other side of street		Full access to both sides of street	Limited access to other side of street
Network Connectivity	Does not address demand for 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 18-12: 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.

18.4.3 Buffered Bike Lanes

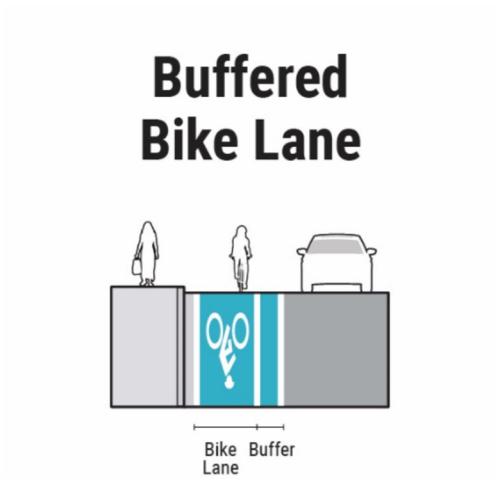


Figure 18-20: Example Buffered Bike Lane Schematic

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 bikeway users.

18.4.3.1 Application

See **Figure 18-3** in **Section 18.2.4** for initial recommendations on when to use a buffered bike lane. The criteria below defines the criteria for all available applications for all speeds.

18.4.3.2 Bike Lane Width

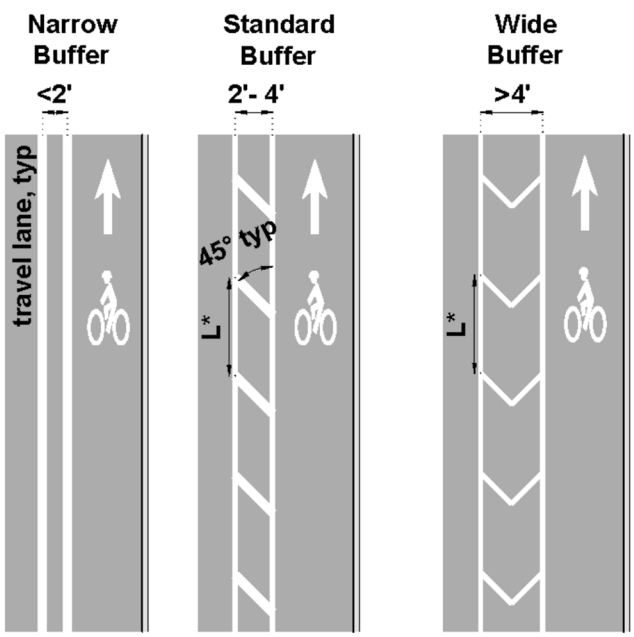
The desirable useable width of a buffered bike lane is 5 to 7-ft exclusive of the buffer. The minimum useable width is 4-ft exclusive of the buffer. The usable width of the bike lane is measured from the outside buffer stripe to either the gutter joint or 1-ft from the nominal face of a monolithic curb.

18.4.3.3 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. Typical spacing is 20-ft with some locations reduces to as low as 5-ft where engineering judgement determines a more frequent spacing is desirable. See **Figure 18-21** for examples of bike buffer lane types.



L = 20' (typical); L = posted speed limit (max)
*spacing may be reduced to as little as 5' where engineering judgement determines more frequent spacing

Figure 18-21: Examples of Buffered Bike Lane Types

18.4.3.4 Signing and Marking

The stripe near the travel lane should be 6-in 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.

18.4.3.5 Bicycle Design Speed

Not applicable as the roadway will be designed to accommodate motorist target design speeds which will exceed bicyclist's design speed.

18.4.3.6 Cross Slope and Grade

Not applicable.

18.4.3.7 Other Considerations

Not applicable.

18.4.4 Bike Lanes

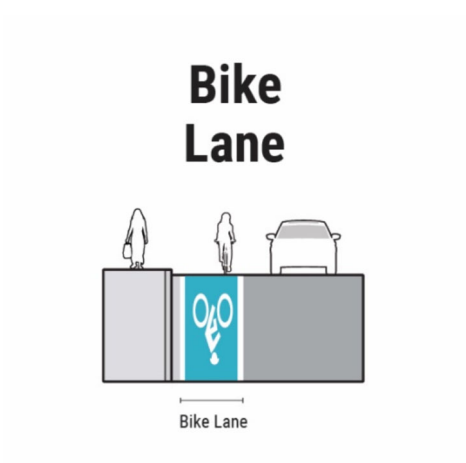


Figure 18-22: Example Bike Lane Schematic

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.

18.4.4.1 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 18.4.5**. This can be particularly beneficial at intersections, see **Section 18.5**.

18.4.4.2 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 18-13** 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 18-13: 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[2,3]	5.5 - 7.5	5
To allow side-by-side bicycling or passing	8 - 10	7.5

[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

[3] Usable width of raised bike lane is measured from back of curb alongside travel lane to edge of bike lane pavement

18.4.4.3 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; and
- 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.

18.4.4.4 Bicycle Design Speed

Not applicable as the roadway will be designed to accommodate motorist target design speeds which will exceed bicyclists.

18.4.4.5 Cross Slope and Grade

Not applicable.

18.4.4.6 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. See **Section 18.3.9** for more information on drainage inlet grates in bicycle facilities.

18.4.5 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 18.4.2**) 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 it can be located at sidewalk height (sidewalk level). Specific information on the curb types adjacent to raised bike lanes are discussed in **Section 18.3.10**.

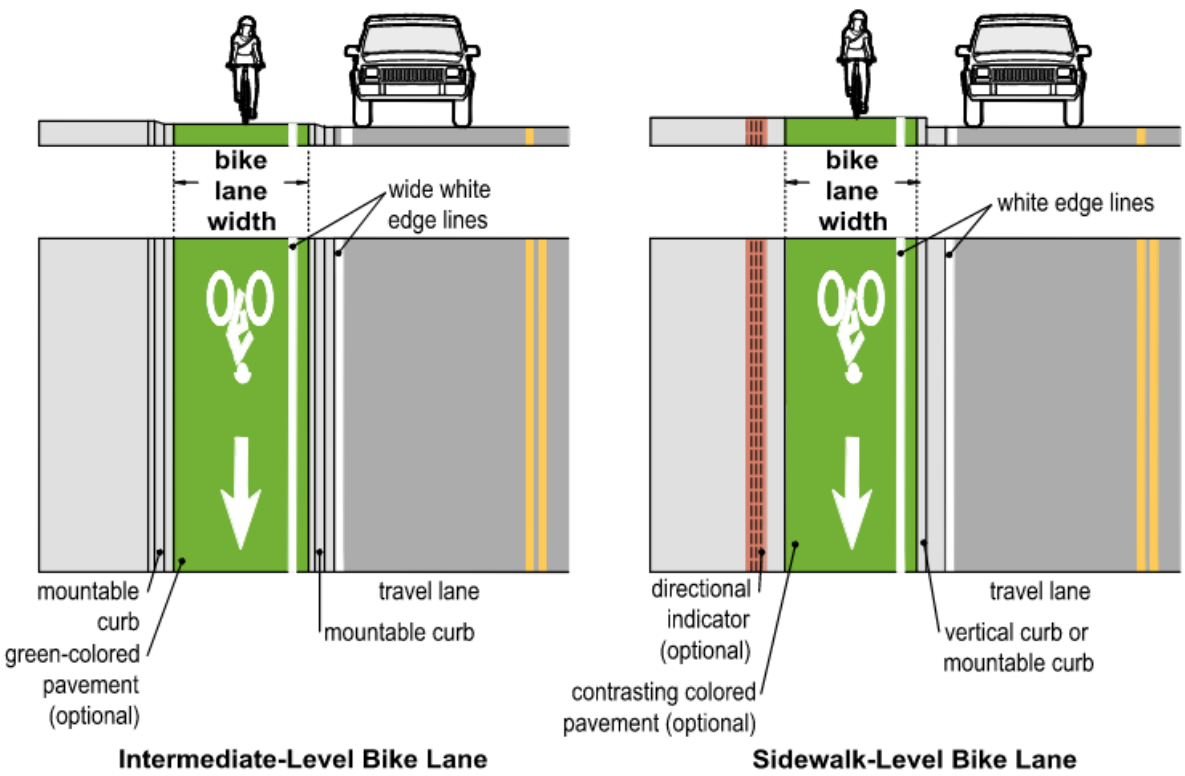


Figure 18-23: Raised Bike Lane Considerations

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.4.5.1 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 18.5.3**).

18.4.5.2 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 18-13 provides minimum and constrained widths for both raised bike lane scenarios.

18.4.5.3 Signing and Marking

A white edge line should be located at the edge of the travel lane to clarify the boundary 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 18-23).

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.

18.4.5.4 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.

18.4.5.5 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) while allowing for adequate conveyance of drainage; however, cross slope may match existing roadway conditions without a maximum where necessary.

18.4.5.6 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 18.4.2.2 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 are discussed in **Section 18.5** 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.

18.4.6 Bike Accessible Shoulders



Figure 18-24: 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 areas for bicyclist and automobile movement, bicyclists may leave the shoulder to pass other cyclists or avoid debris and other traffic conflicts.

18.4.6.1 Application

See **Figure 18-4** for guidance on recommended shoulder width. In rural environments, see **Figure 18-4** for guidance on desirable shoulders widths.

18.4.6.2 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 4** 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 18-4**.

Some shoulders should be up to 10-ft wide adjacent to higher speed roadways as indicated in **Figure 18-4** to allow bicyclists to operate with more separation to the high-speed traffic. Roadways indicated in **TxDOT's Bicycle Tourism Trails Study** 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. Contact TxDOT's Public Transportation Division (PTN) for more information. The minimum widths for a bike accessible shoulder are defined below.

- A minimum width of 4-ft is allowable in low speed (45 mph or less) conditions;
- A minimum width of 5-ft is allowable for high-speed conditions; and
- A minimum width of 5-ft 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.

18.4.6.3 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.

18.4.6.4 Bicycle Design Speed

Not applicable as the roadway will be designed to accommodate motorist target design speeds which will exceed bicyclists.

18.4.6.5 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) while allowing adequate conveyance of drainage; however, cross slope may match existing roadway conditions without a maximum where necessary.

18.4.6.6 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 18-25**). Refer to the latest RS Standards for additional guidance on rumble strips.

Periodic gaps (**Figure 18-26**) 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; and
- 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).

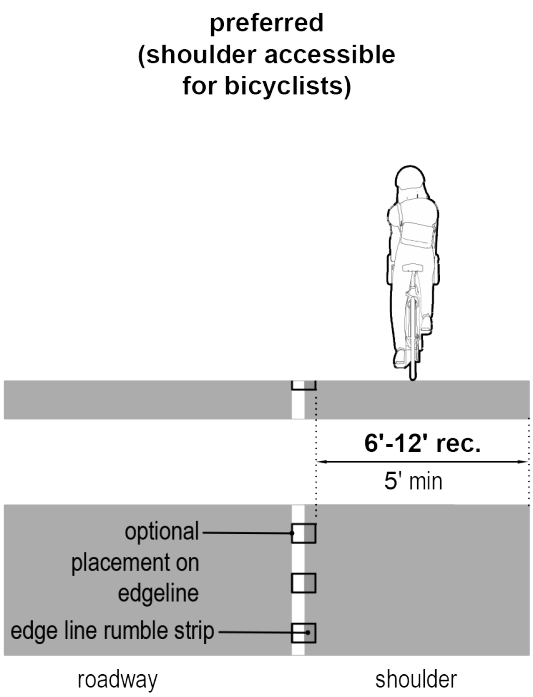


Figure 18-25: Rumble Strip Placement in a Shoulder

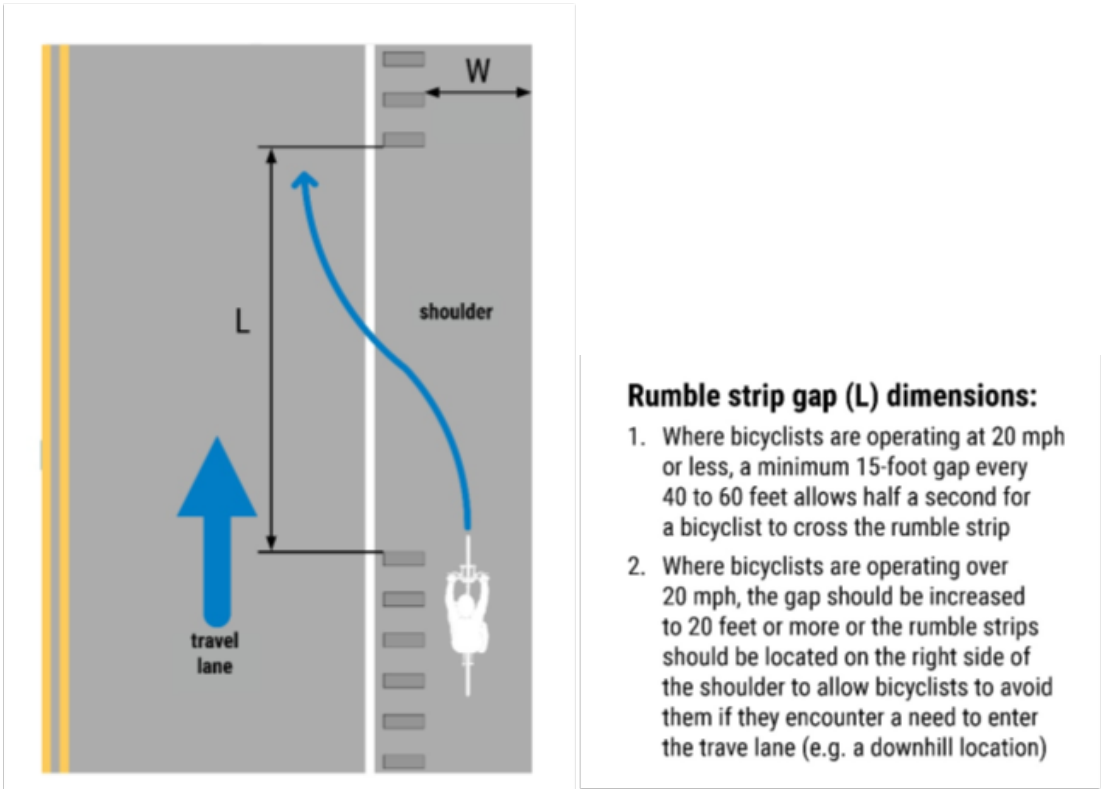


Figure 18-26: Rumble Strip Design and Gap Placement

18.4.7 Shared Lanes (wide outside lane)

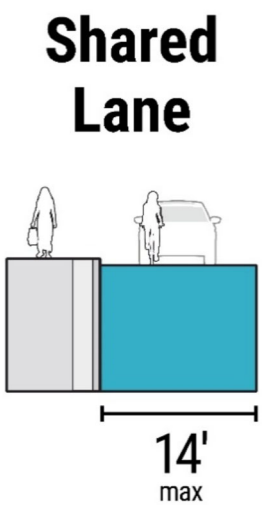


Figure 18-27: Example Shared Lane Schematic

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.

18.4.7.1 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).

18.4.7.2 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.

18.4.7.3 Signing and Marking

Typical supplemental signage may include: BICYCLES MAY USE FULL LANE (R4-11). Refer to the **TMUTCD** for signing applications.

Custom signage may include language instructing vehicles to change lanes to pass or use a 3-ft passing distance.

18.4.7.4 Bicycle Design Speed

Not applicable as the roadway will be designed to accommodate motorist target design speeds which will exceed bicyclists.

18.4.7.5 Cross Slope and Grade

Not applicable.

18.4.7.6 Other Considerations

Lighting can be a beneficial element for Shared Lane facilities. It can significantly enhance safety and visibility for cyclists, particularly at night or in low-light conditions. Effective intersection lighting should be prioritized when considering lighting as a part of this facility. It can assist all users to navigate the intersection

safely and confidently, reducing the risk of conflicts.

18.4.8 Rural Bikeway Types

The following bikeway types are the most common in rural areas. Other bikeway types may be considered where a higher degree of comfort is desired for bicycling following the guidance of **Figure 18-14**. For example, On the **Texas Bicycle Tourism Trail Example Network**, other bikeway types, such as a buffered bike lane, may be desirable in place of a shoulder.



Figure 18-28: Example Photo of Rural Shared Use Path

18.4.8.1 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 18.4.1** and the **AASHTO Bike Guide** for further design guidance.



Figure 18-29: Example Rural Bike Accessible Shoulder Schematic

18.4.8.2 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 18.4.6**, and the **AASHTO Bike Guide** for further design guidance.

18.4.8.3 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.

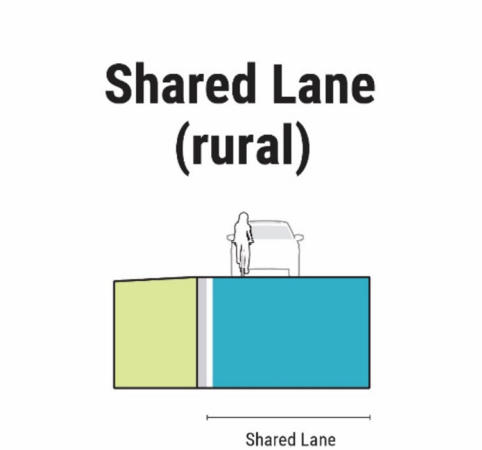


Figure 18-30: Example Rural Shared Lane Schematic

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.

18.5 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.

A bicycle crossing is any location where a bicyclist enters and crosses a roadway from one dedicated bikeway to another. Bikeway crossings of roads can be broadly categorized as mid-block, intersection, or grade separated.

Intersection crossings occur within the functional area of an intersection of two or more roadways (see **Figure 18-31**). 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 visibility 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.

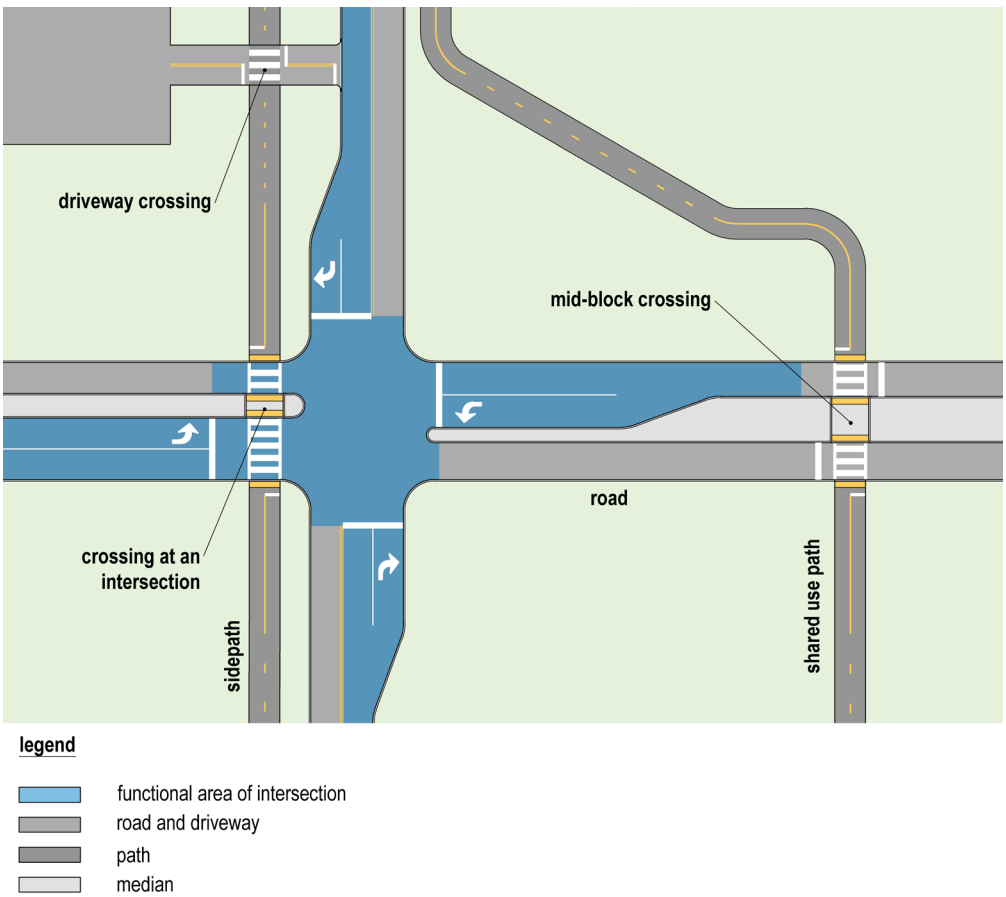


Figure 18-31: Crossing Locations Relative to Intersections Functional Area

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.1 Principles of Intersection Design

18.5.1.1 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.

A primary concern for safety and comfort on facilities where bicyclists and pedestrians operating in shared space is the relatively higher speeds bicyclist operate compared to pedestrians. To mitigate crash risk between bicyclists and pedestrians and improve comfort for both users it is important to:

- Provide clear sight distance between pedestrians and approaching bicyclists at locations where bicyclists cross a pedestrian facility (see **Section 18.3.5**);
- Provide facilities with widths which can accommodate the volume of users; or
- Separate bicyclists from pedestrians.

18.5.1.2 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.

18.5.1.3 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;
- 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.

18.5.1.4 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, react to, and avoid potential conflicts. See **Section 18.3.5** for more details.

18.5.1.5 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. Transitions from off-street bikeways to on-street facilities should be clearly communicated with advance warning signs to bicyclists and in some instances,

motorists where bicyclists entering the roadway may be unexpected. Advance warning signs or pavement markings should be considered at locations where a bikeway may unexpectedly narrow at a transition. Transitions can also benefit from the provision of wayfinding to guide users along the preferred path of travel. It is also important to provide clear and direct paths for pedestrians across bikeways. Protected intersection designs may reduce the likelihood that pedestrians will use a bike lane as a walkway. See **Section 18.5.2** for more details.

18.5.1.6 Accommodate people with disabilities

Intersections must 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 19** and other general information in the pedestrian guidance.

18.5.2 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 recognizable 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 18.5.3**) should be considered where the bikeway selection guidance in **Section 18.2.4** indicates a need for additional separation and the existing condition is a shared lane, shoulder, or bicycle lane.

The following treatments are not presented in order of priority. The bikeway selection guidance of **Section 18.2.4** should be used when evaluating intersection treatments in addition to the principals of **Section 18.5.1**. It is preferable to provide the same level of accommodation as mid-block sections of a roadway.

18.5.2.1 Shoulder or Bicycle Lane Terminated to a Shared Through Lane

Figure 18-32 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 with 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.

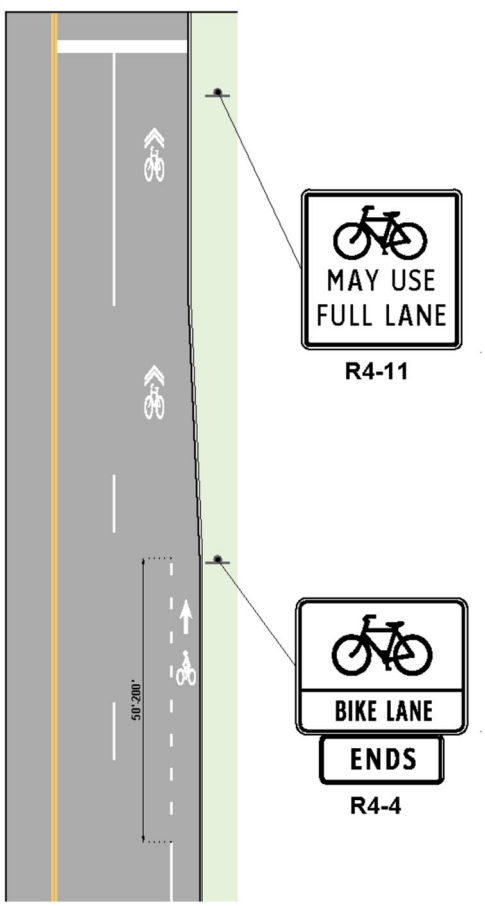


Figure 18-32: Example Shoulder or Bike Lane Terminating to a Shared Through Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.2 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 18-33**);
- Bike lane transitions to a shared through lane (**Figure 18-34**);
- Bike lane transitions to a protected intersection (**Figure 18-40**) or sidewalk via a bike ramp.

Shared lane markings may be located within the left side of the turn lane (**Figure 18-33**) if the lane is a minimum 14-ft, 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 18-34**) following **TMUTCD** guidance.
At locations where the lane is 15 or more feet in width, provision of a bicycle lane should be considered.

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.

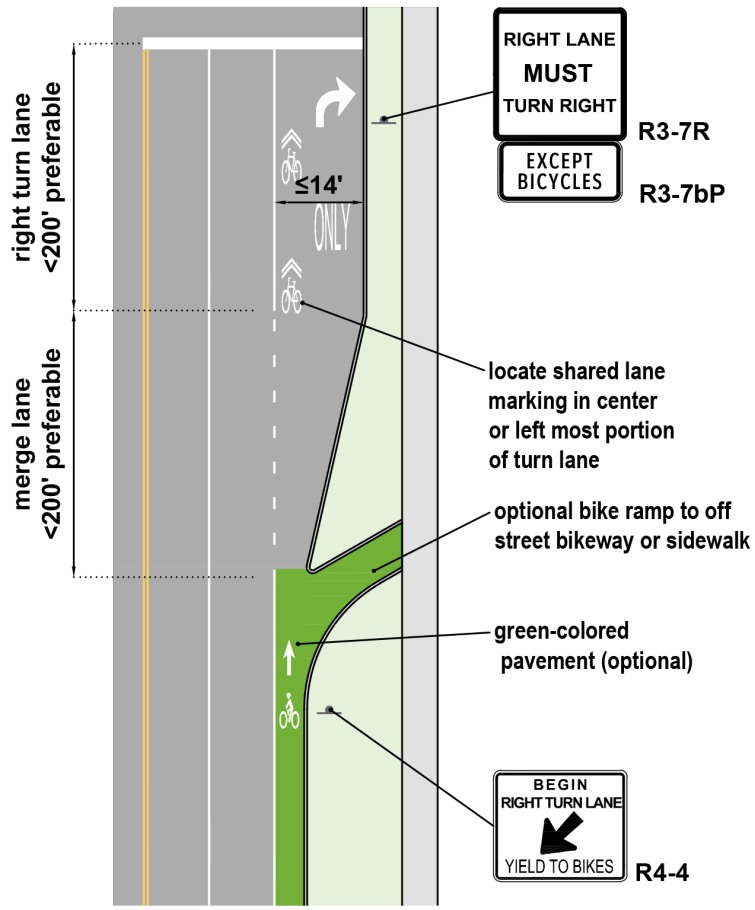


Figure 18-33: Example Bike Lane Transitions to a Shared Lane Markings in Right Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

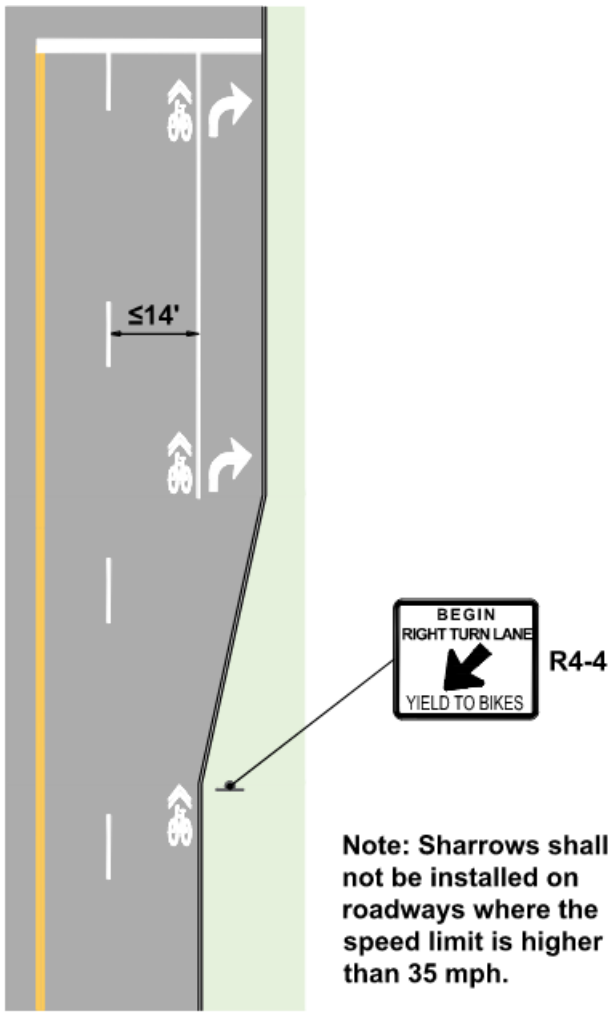


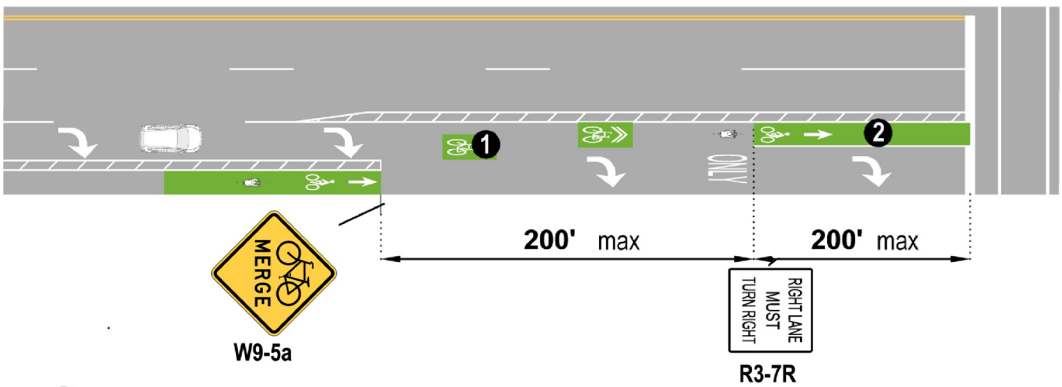
Figure 18-34: Example Bike Lane Transitions to a Shared Lane Markings in Through Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.3 Shoulders and Bicycle Lanes Continue with Addition of Right Turn Lanes

Figure 18-35 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 30 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.



1. Note: - Optional shared lane markings if posted speed ≤30 mph, the shared lane markings are appropriate to assist bicyclists with positioning, with or without a bicycle lane at the intersection.
2. Green-colored pavement (optional)

Figure 18-35: Bike Lane Transitions to Through Bike Lane with Right Turn Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

Figure 18-36 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.

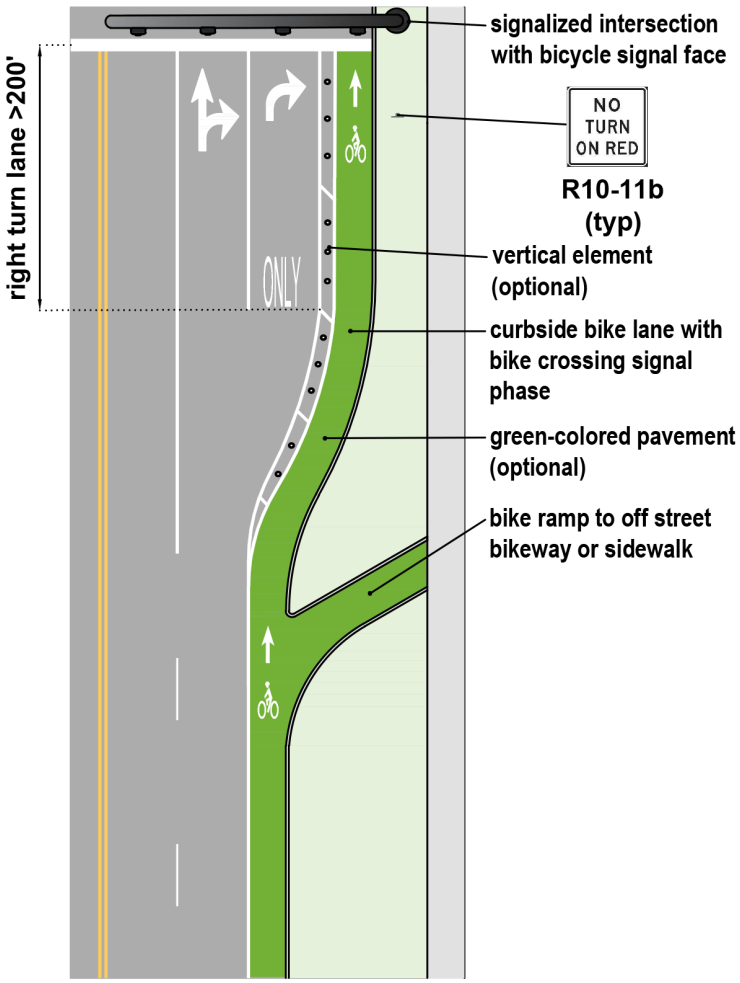


Figure 18-36: Example Bike Lane Approach to a Right Turn Only Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

Figure 18-37 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 18-37**).

Figure 18-38 provides additional design options to accommodate bike lanes as they approach an intersection.

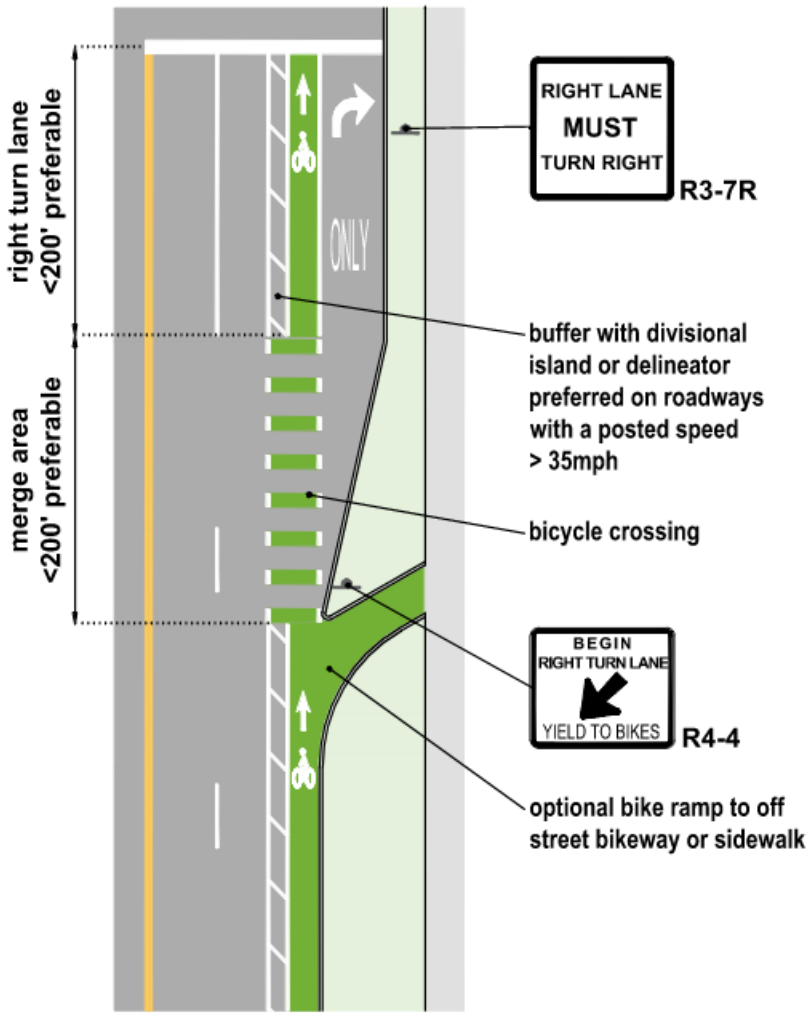


Figure 18-37: Bicycle Lane with Constricted Motorist Entry Point

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

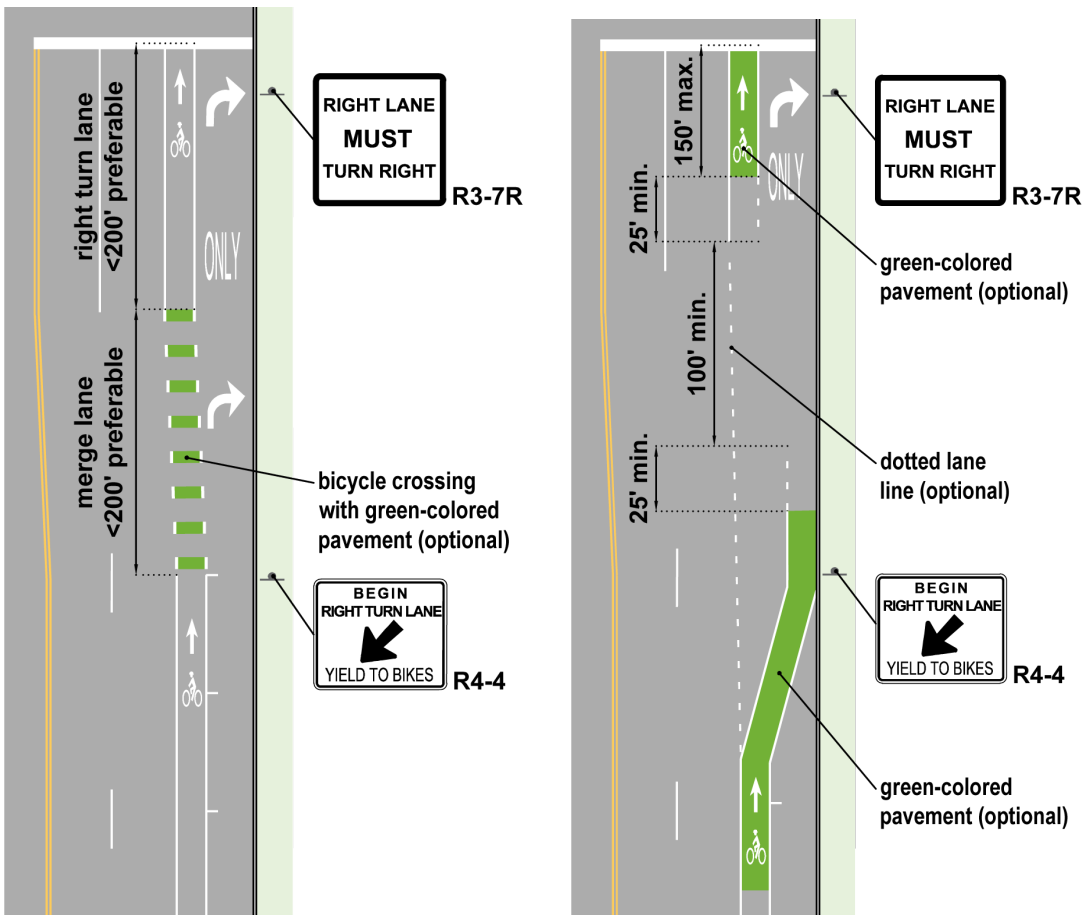


Figure 18-38: Standard Bicycle Lane with Right Turn Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.4 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 ROW at the intersection as through moving vehicles, and then transition back to a paved shoulder on the far side of the intersection (**Figure 18-39**). 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 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 18-39**). 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.

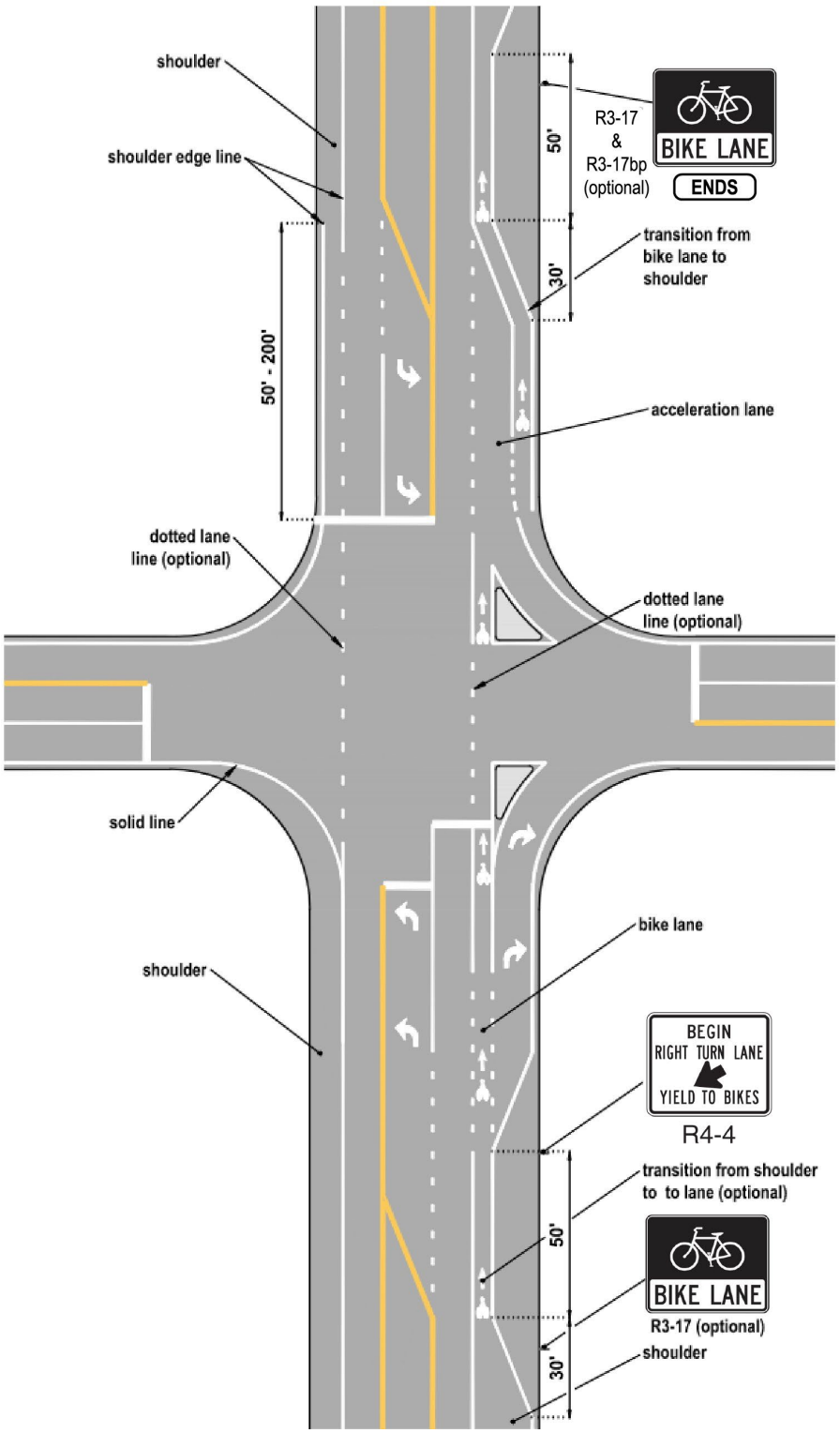


Figure 18-39: Example Shoulder Markings to Accommodate Bicycling

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.5 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 18-40**). Where this is done on a roadway with paved shoulders, at least 4-ft of usable shoulder pavement clear of rumble strips should be carried through the intersection along the outside of the bypass lane and designated as a bike lane. At locations where guardrails are present, an additional 2-ft of shoulder width is recommended. 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 the bypass lanes.

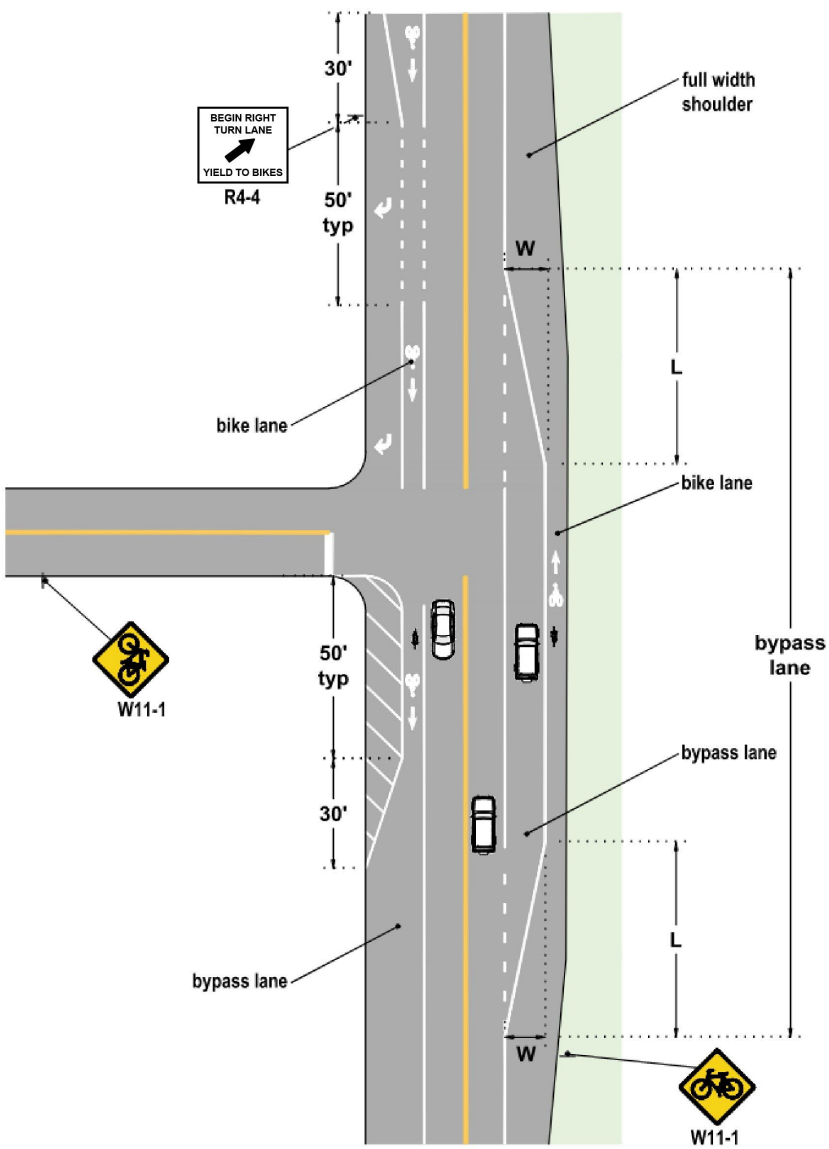


Figure 18-40: Motorist Bypass Lane with Bicycle Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.6 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 18-41**). 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 intersections 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.

Additional guidance on protected intersections may be found at:

- **Don't Give Up at the Intersection, National Association of City Transportation Officials (NACTO)**
- **Protected Bikeways Practitioners Guide, Institute of Transportation Engineers**
- **Separated Bike Lane Planning and Design Guide, FHWA**
- **Separated Bike Lanes on Higher Speed Roadways: A Toolkit and Guide, FHWA**

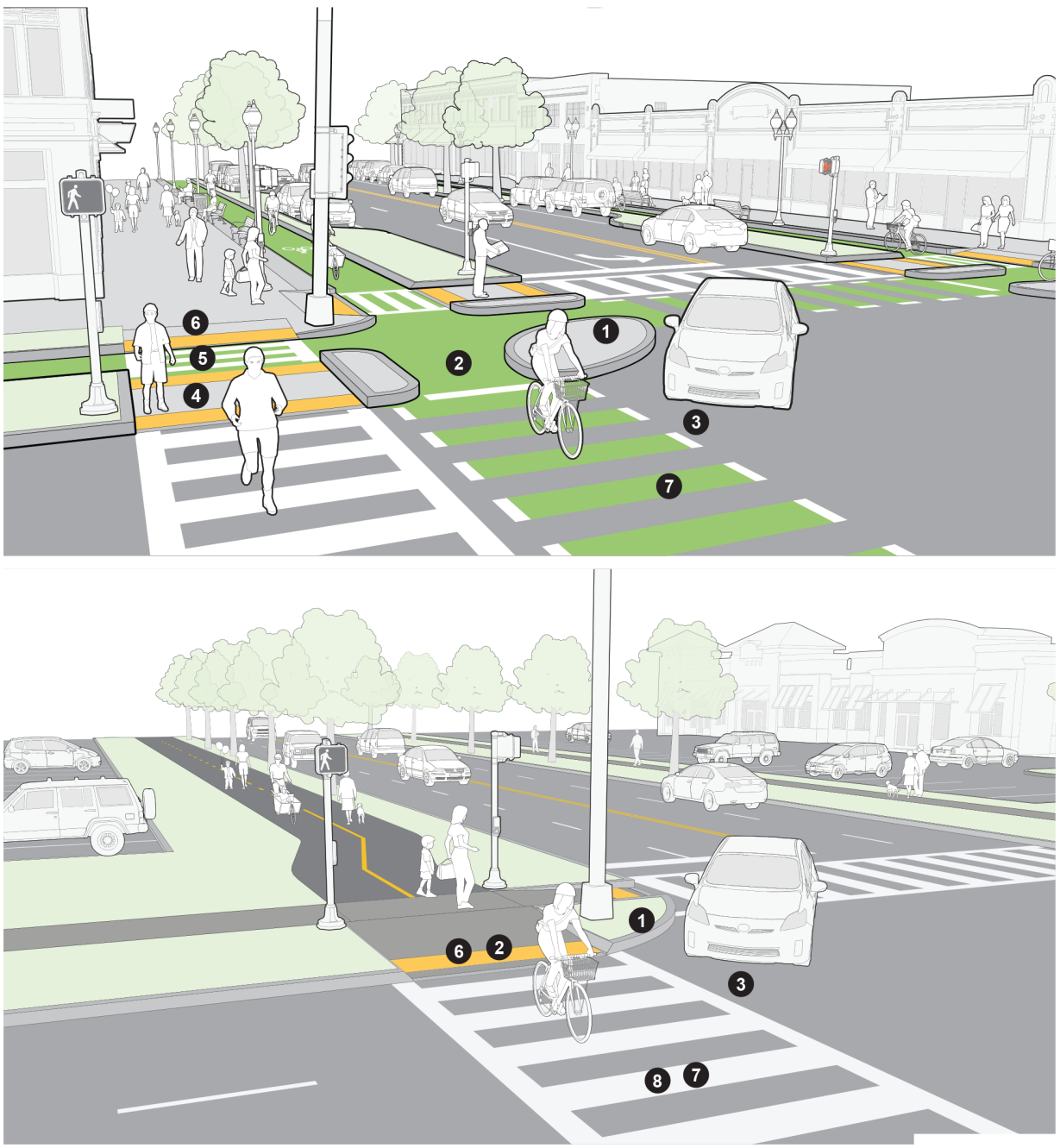


Figure 18-41: Protected Corner Treatment and Intersection Design Components

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.6.1 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. The corner island should be constructed with a standard vertical curb to discourage and prevent motor vehicle encroachment. In retrofit projects, 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 18-42** 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.

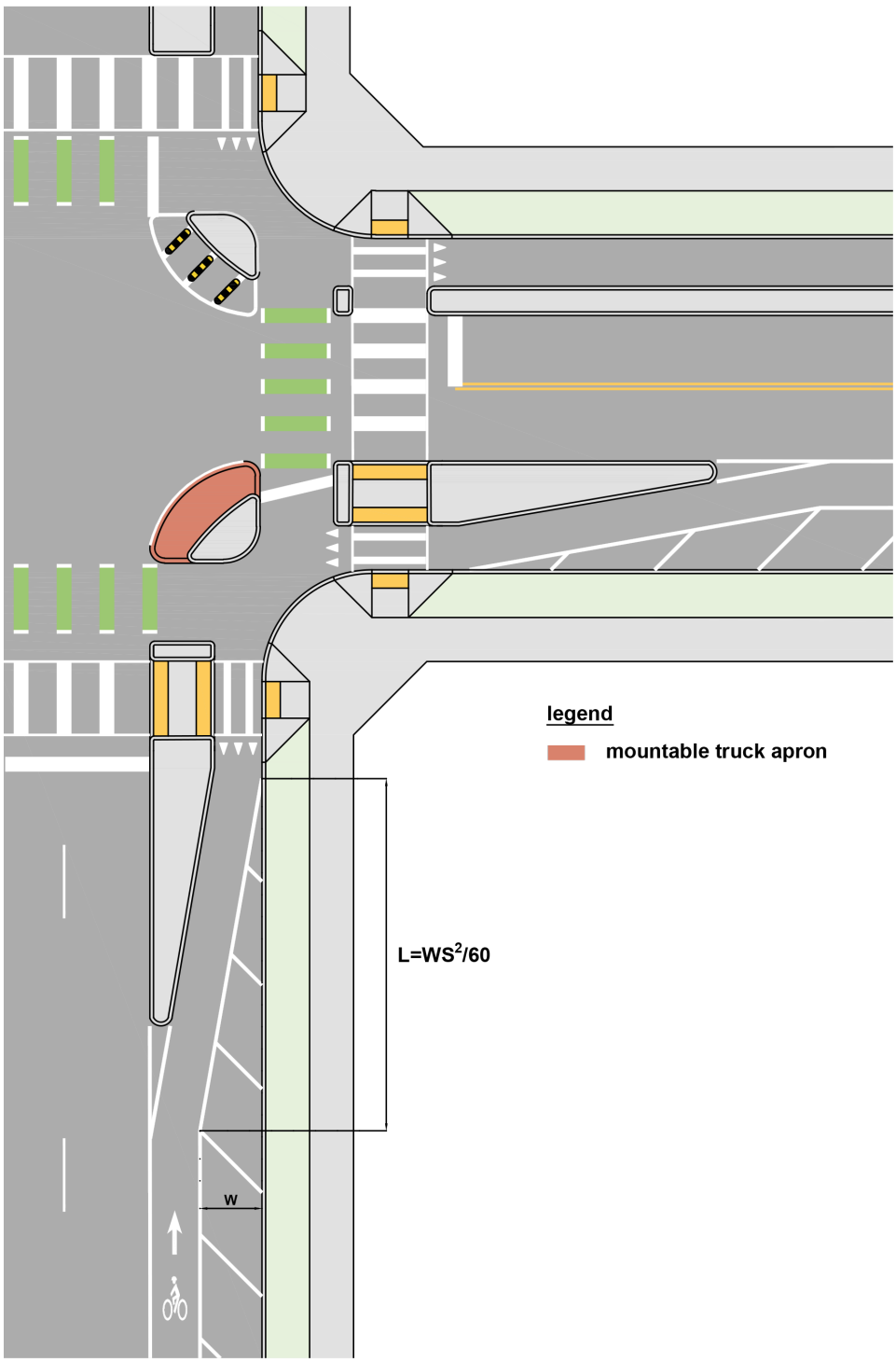


Figure 18-42: Protected Intersection Treatments with Truck Apron

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.6.2 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 18-43** 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 (a.k.a., stop line) is located behind the larger vehicle turning path.

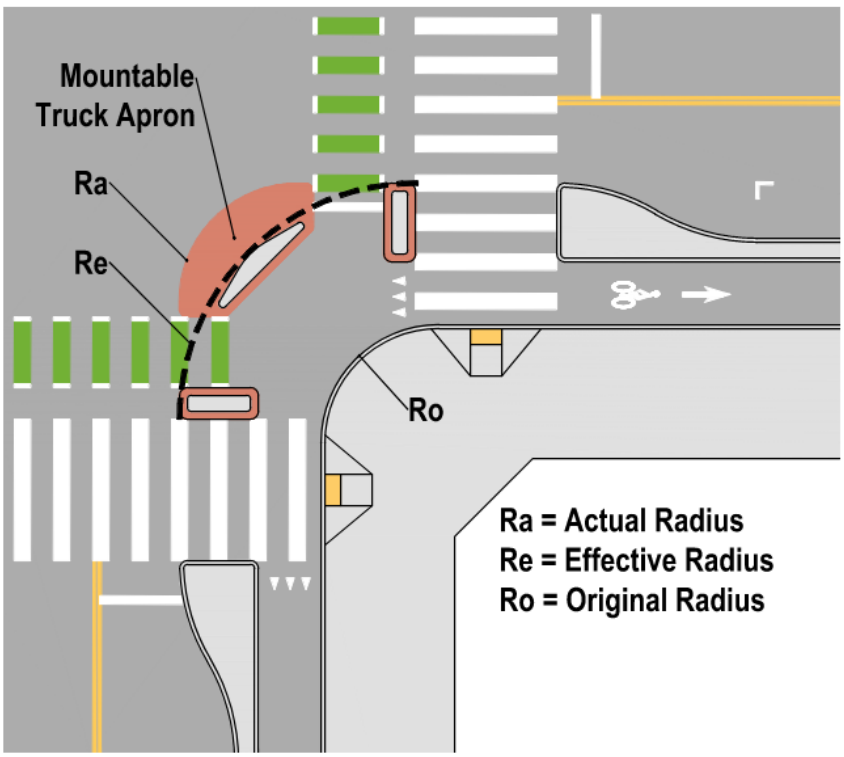


Figure 18-43: Actual vs Effective Radius

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.6.3 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. **Figure 18-44** below details various protected corner treatment methods.

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.

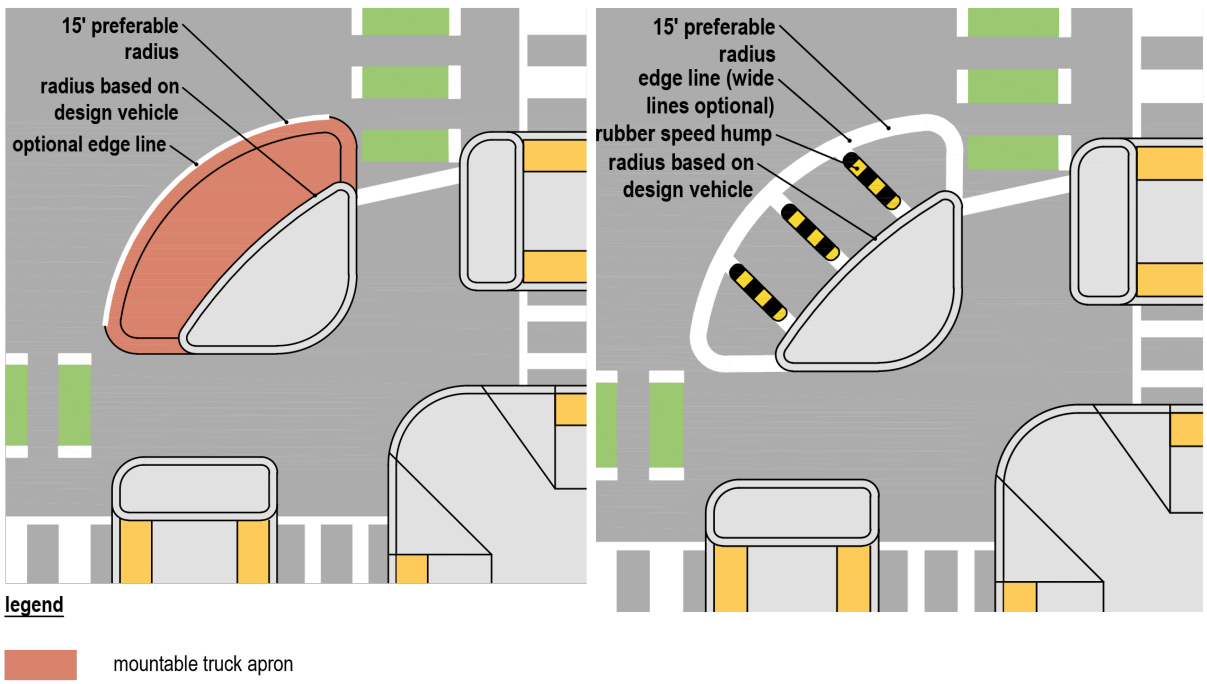


Figure 18-44: Protected Corner Treatment Details: Truck Apron (left) and Turning Wedge (right)

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.6.4 Traffic Control Considerations

At all intersections, the designer should consider using a TURNING VEHICLES YIELD TO PEDESTRIANS AND BICYCLES (R10-15) 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.

18.5.2.7 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.

18.5.2.7.1 Transition from Separated Bike Lane to Shared Lane

It is preferable for the transition into a shared lane to occur after crossing a 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 18-45**). Where this is not an option, the transition on the approach to the intersection should be in advance of the stop line on the roadway.

This location should ensure a clear sight line between all approaching motorists and the bicyclist. It may be desirable to add a BICYCLES MAY USE FULL LANE (R4-11) 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.

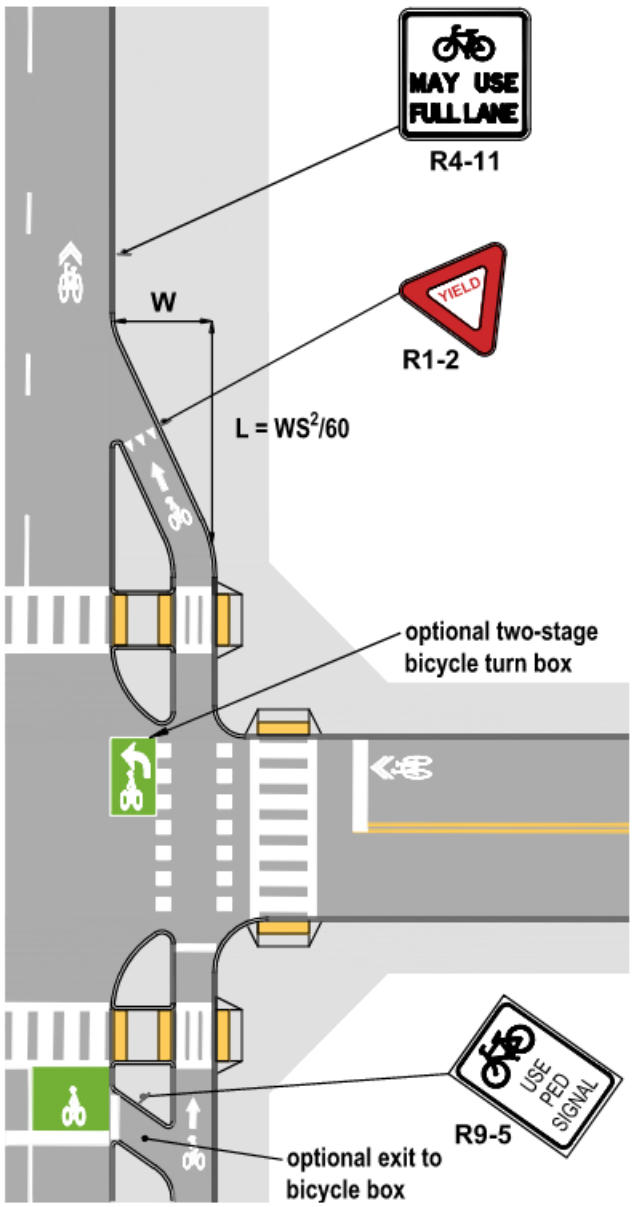


Figure 18-45: Transition to Shared Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.7.2 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 18-46**). This will minimize the pedestrian crossing of the roadway.

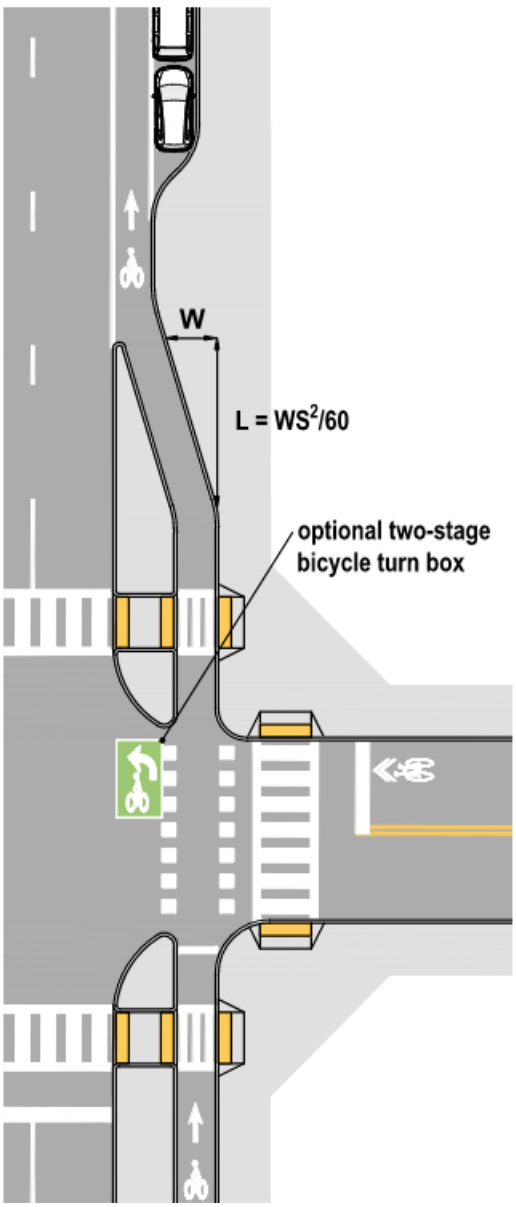


Figure 18-46: Transition to Bike Lane

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.7.3 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 18-47** and **Figure 18-48**. The crossing may warrant bicycle signals at signalized crossings (see **Section 18.3.12**). The use of directional, tapered

islands can provide positive direction for bicyclists to follow the desired transition route. It may also be desirable to use green-colored pavement within crossings and two-stage bicycle turn boxes to improve legibility and provide strong visual guidance of the intended path across the intersection to all users. The crossing may warrant bicycle signals at signalized crossings. The signal should be coordinated with the intersecting street signal phase.

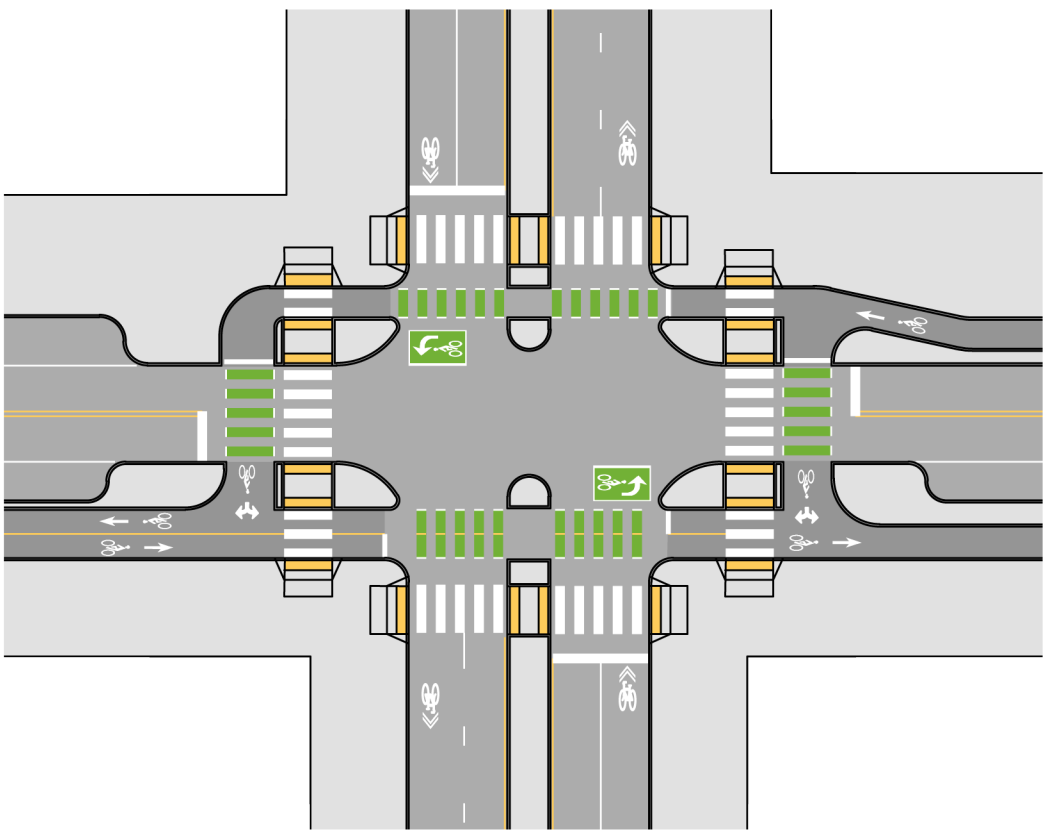


Figure 18-47: Transition from One-Way to Two-Way Separated Bike Lanes with Protected Intersection

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

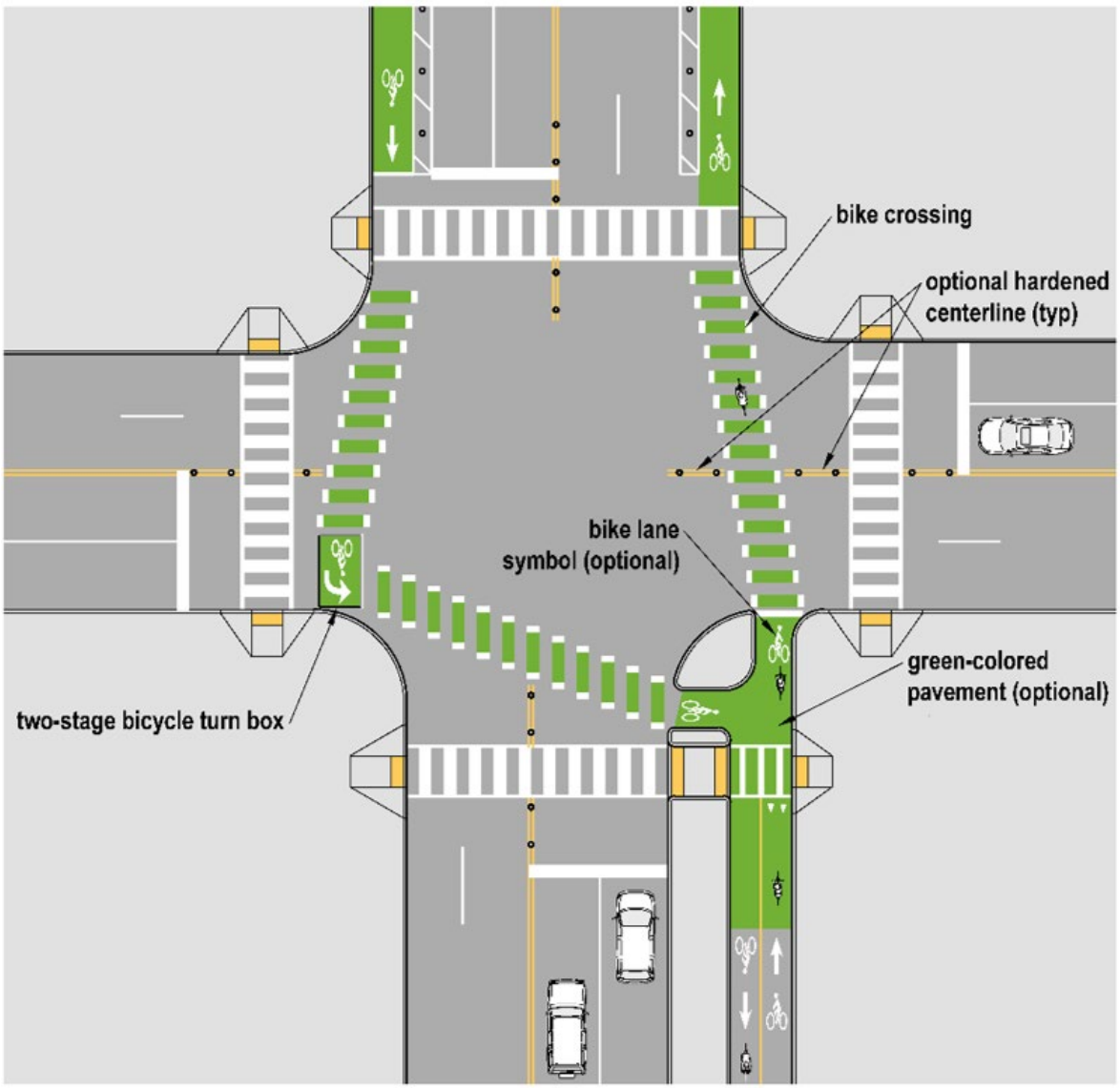


Figure 18-48: Transition from One-Way to Two-Way Separated Bike Lanes with Protected Corner and Two-Stage Turn Queue Box

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.2.8 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 18-49**). 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. From the point where the bicycle lane shifts to become a separated bike lane, the design should follow the Protected Intersection Guidance in **Section 18.5.2.6** (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) or locations where a phase separated right turn lane is provided, 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.

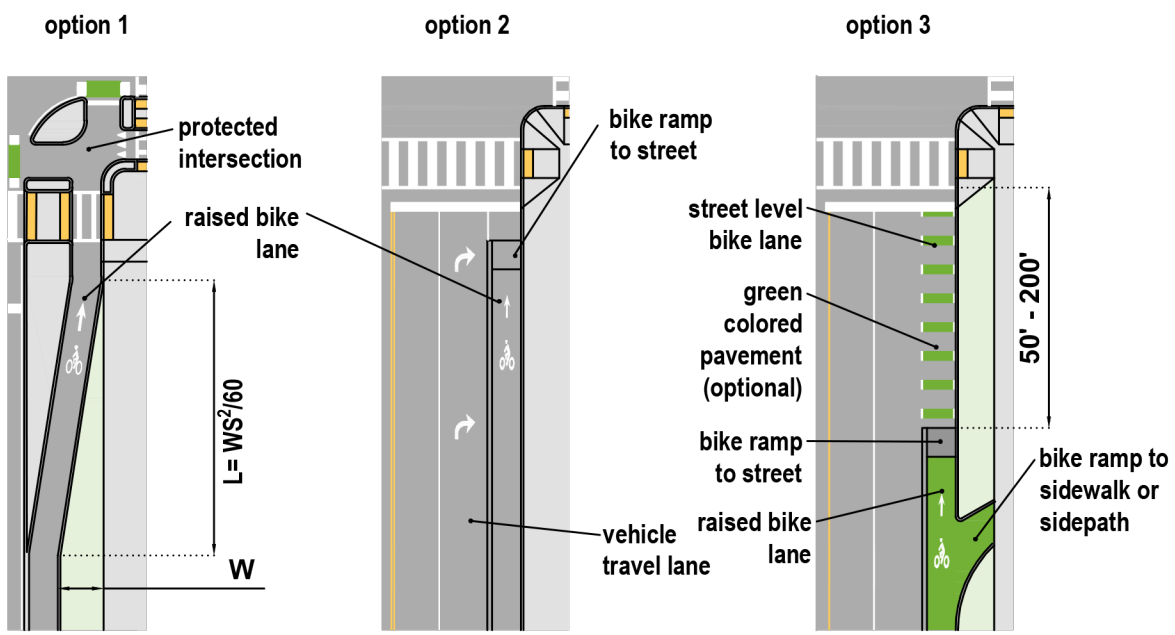


Figure 18-49: Raised Bike Lanes at Intersections

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.3 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 interchange ramps, areas with heavy weaving or merging, intersections with high turning volumes, or the approaches to and departures from roundabouts.

It is preferable to design bike ramp transitions to occur with a gradual taper as depicted in Detail 1 of **Figure 18-50** following the guidance of **Section 18.5.3**. This is more feasible at

locations with a wider buffer with a ramp width of 6-ft to 8-ft recommended.

In constrained conditions, where a buffer is narrower, a more abrupt shift as depicted in Detail 2 of **Figure 18-50** may be considered or a parallel ramp may be necessary where a buffer is non-existent (Detail 3). These constrained bike ramps can present the following challenges:

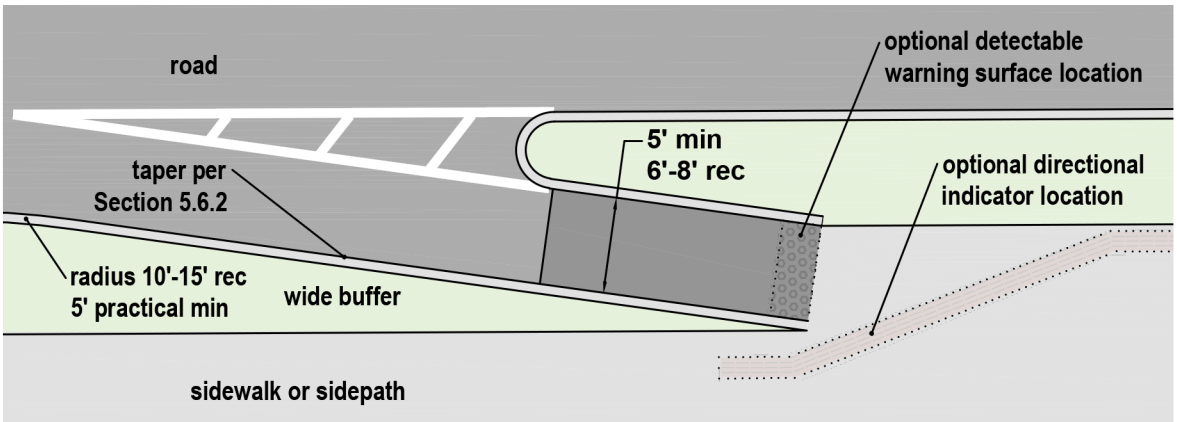
- Narrow bike ramp widths less than 8 ft in width with abrupt approach angles can force bicyclist to encroach on adjacent motorist travel lanes, pedestrian zones, or on-coming bicycle traffic on two-way facilities in order to navigate the ramp; and
- If grade breaks at the top and bottom of the bike 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 and will need to slow significantly to navigate the bicycle ramp.

In constrained buffer situations, a minimum ramp width of 8-ft can help to address these issues. However, consideration should be given to the potential for the ramp to appear like a driveway to motorists. Where driver confusion is likely, it may be necessary to add design elements which clarify that the access is only intended for bicyclists. The provision of bicycle lane symbols, buffers, or green colored pavement may be considered. Where hardened restrictions are necessary, the guidance for restricting motorist access onto shared use paths in **Section 18.3.14** should be followed.

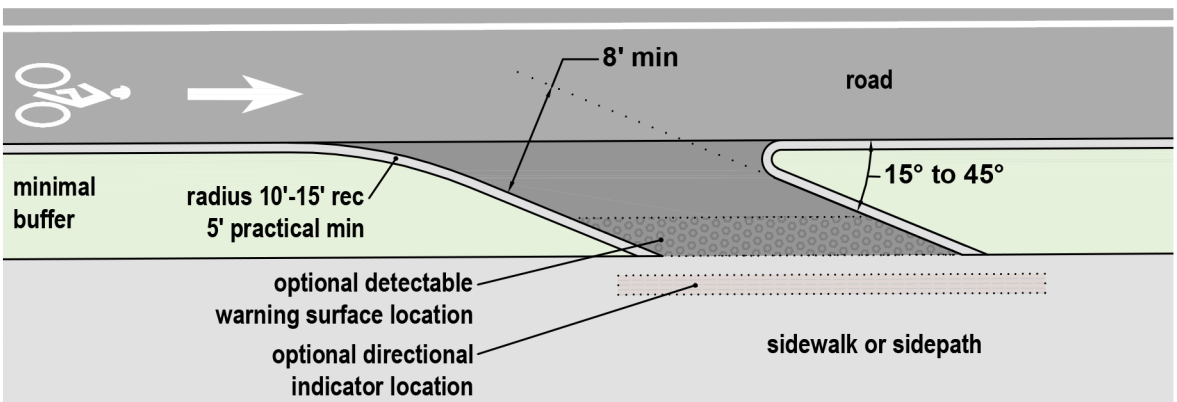
Bike ramps are intended for the exclusive use of bicyclists and therefore need not comply with pedestrian accessibility guidelines but grades similar to pedestrian curb ramps can help to address issues of bicyclist comfort.

Where the bike ramp connects directly to a sidewalk or shared use path, a detectable warning surface, a directional indicator surface, or both may be used at the top of the bike ramp to warn pedestrians and/or guide pedestrians away from the ramp as shown in **Figure 18-50**.

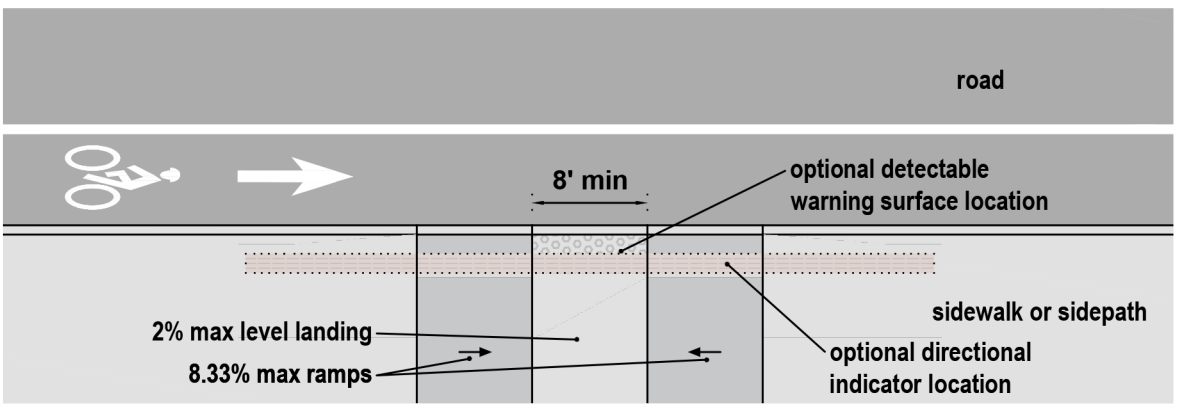
A directional indicator is a tactile surface comprised of raised bars detectable underfoot and with a white cane that may be deployed parallel to the pedestrian path of travel to help pedestrians follow an accessible pathway or navigate a large open space. They may also be deployed across the pedestrian path of travel to provide guidance to pedestrians with disabilities regarding when to turn (e.g., to locate a mid-block curb ramp, bus island, or transit door).



Detail 1 - preferred bicycle ramp alignment with wide sidewalk buffer



Detail 2 - lateral shift bicycle ramp alignment with minimal width sidewalk buffer



Detail 3 - parallel bicycle ramp with no sidewalk buffer

Figure 18-50: Bicycle Ramps

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.4 Driveways

18.5.4.1 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 18.3.12**); 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 18.3.12**); These driveways may be designed according to the intersection design principles laid out in this chapter. Traffic control should be evaluated with an engineering study.

18.5.4.2 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 19** and **Figure 18-51** 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.

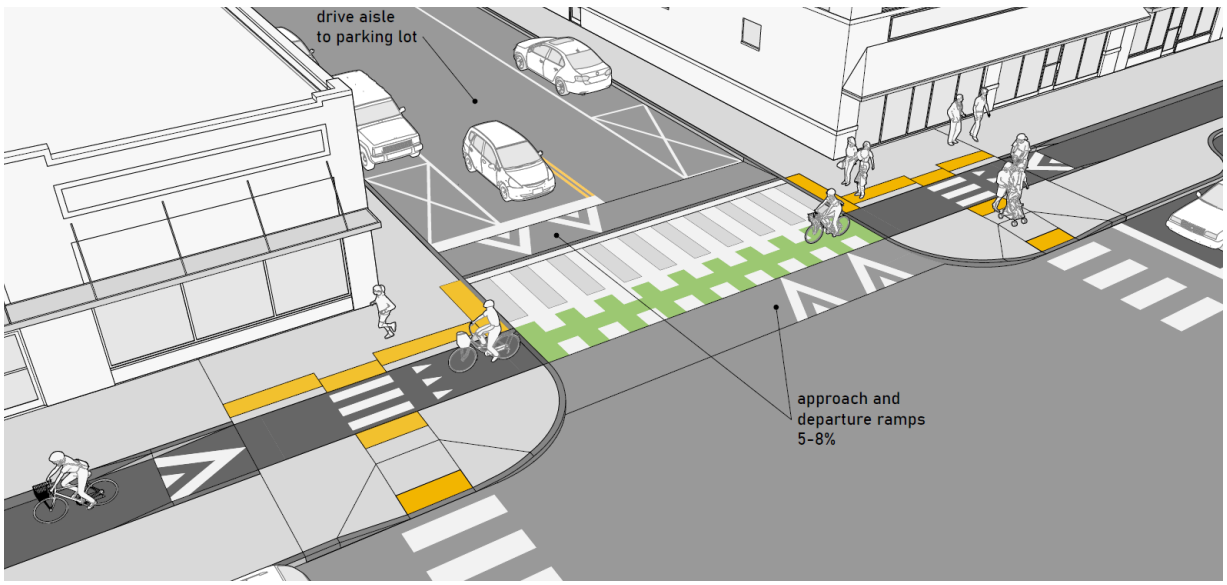


Figure 18-51: Raised Driveway Crossing

(Refer to the **TMUTCD** and **Section 18.1.1** 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 and pedestrians and motorists entering the driveway and 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 previously, **Section 4.11.4** establishes recommended sight triangles that will generally result in sufficient sight distance for bicyclists.

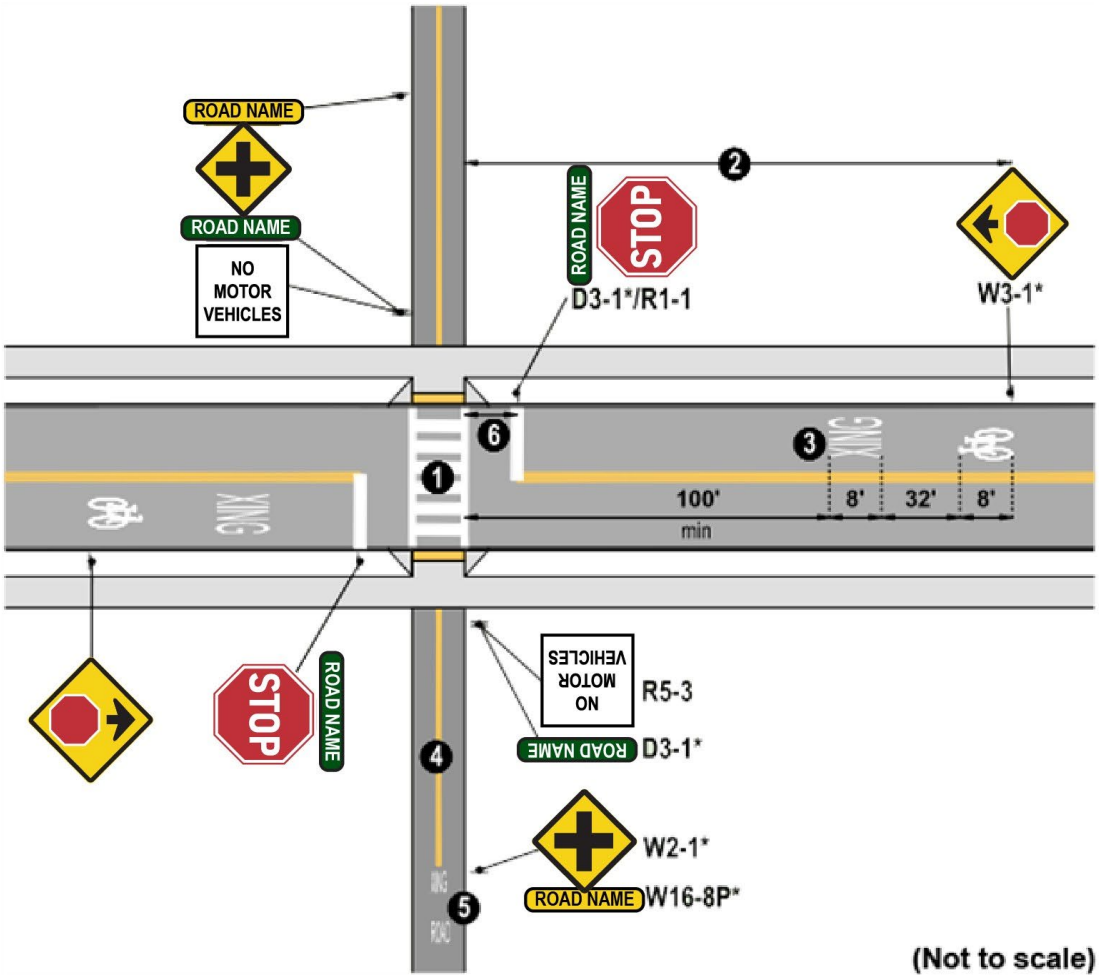
18.5.5 Mid-Block Shared Use Path Crossings

It is preferable for mid-block crossings to intersect the roadway at a 90 degree angle to minimize the crossing distance and maximize sight distance. Mid-block crossings 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-block 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 Safe Transportation for Every Pedestrian (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 18-52 through **Figure 18-54** 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. Refer to the **FHWA STEPS** guide for recommended treatments and countermeasures to address motorist stopping and yielding.

Refer to the **AASHTO Bike Guide** for additional guidance for shared use path crossings.



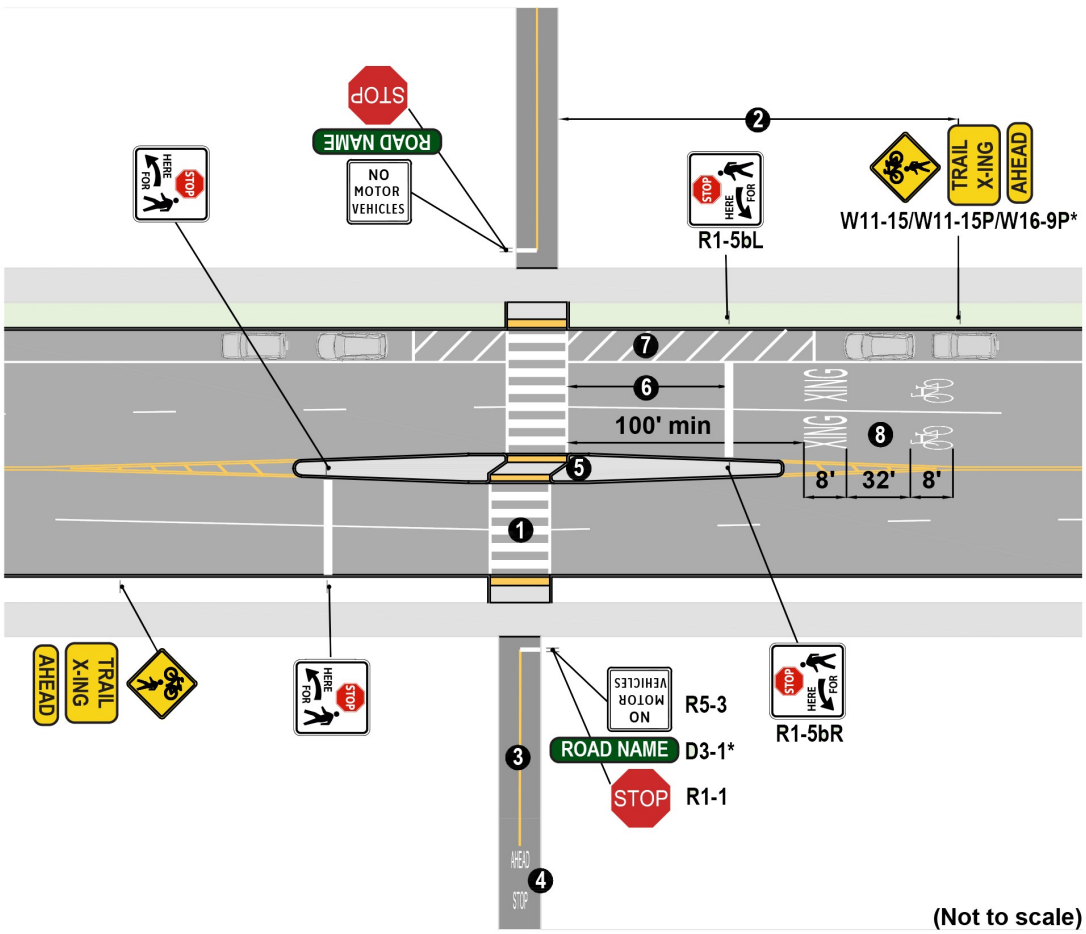
Note:

* 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 18-52: Road Stops

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)



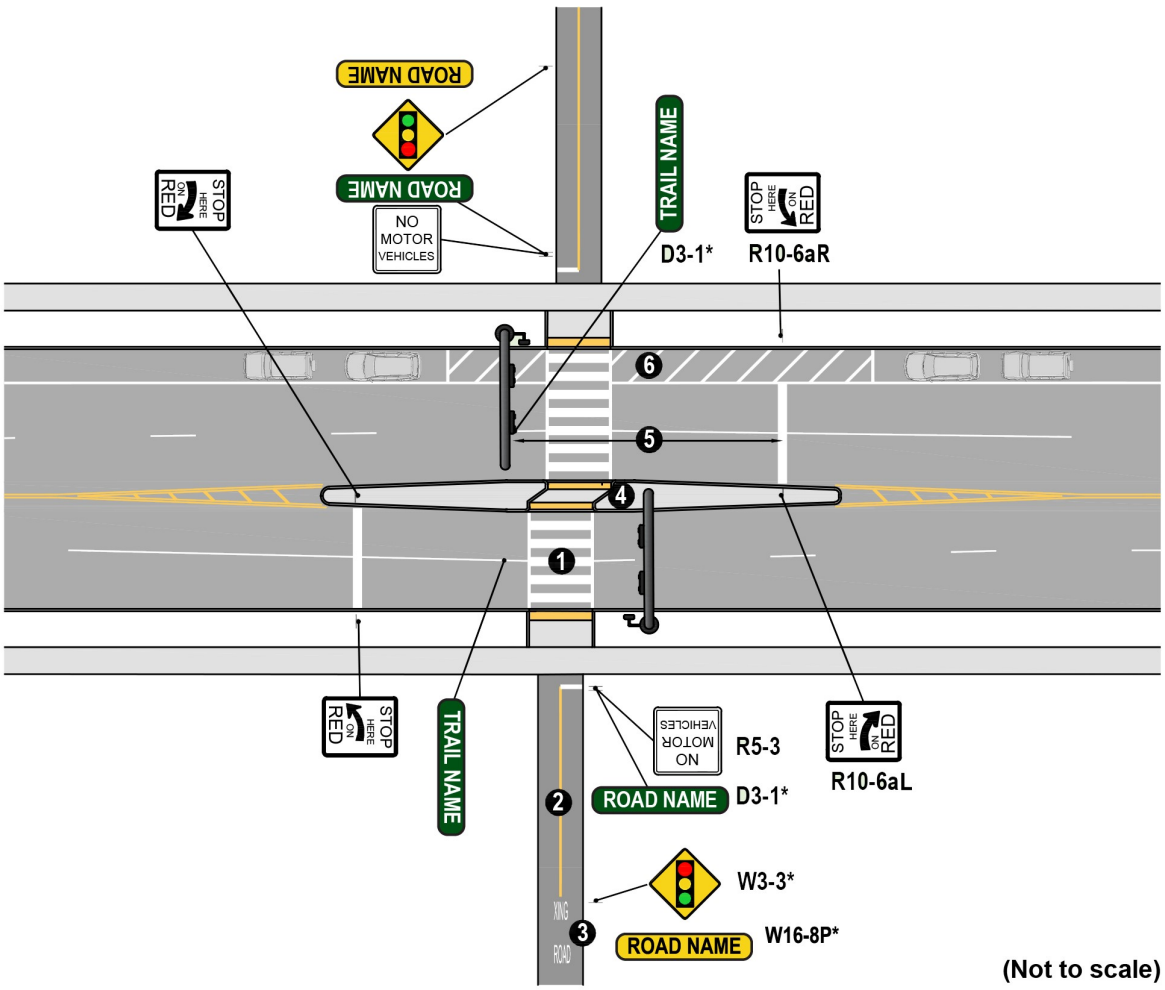
Note:

* **Signs are Optional but Recommended**

1. crosswalk markings legally establish midblock pedestrian crossing
2. length varies: see Texas MUTCD table 2C-4
3. shared-use path centerline as needed
4. optional pathway markings and signage
5. refuge median (6' minimum width)
6. stop bar placement 20'-50'
7. parking restricted
8. optional roadway markings

Figure 18-53: Multilane Road Uncontrolled with Advance Stop Bar Line

(Refer to the **TMUTCD** and **Section 18.1.1** 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 (6' minimum width)
5. stop bar placement 40' min from traffic signal
6. parking restricted

Figure 18-54: Multilane Road with Signals

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.5.6 Roundabouts

Section 14.4.5 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 18-55** and **Figure 18-56**). 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 18-55**, 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.

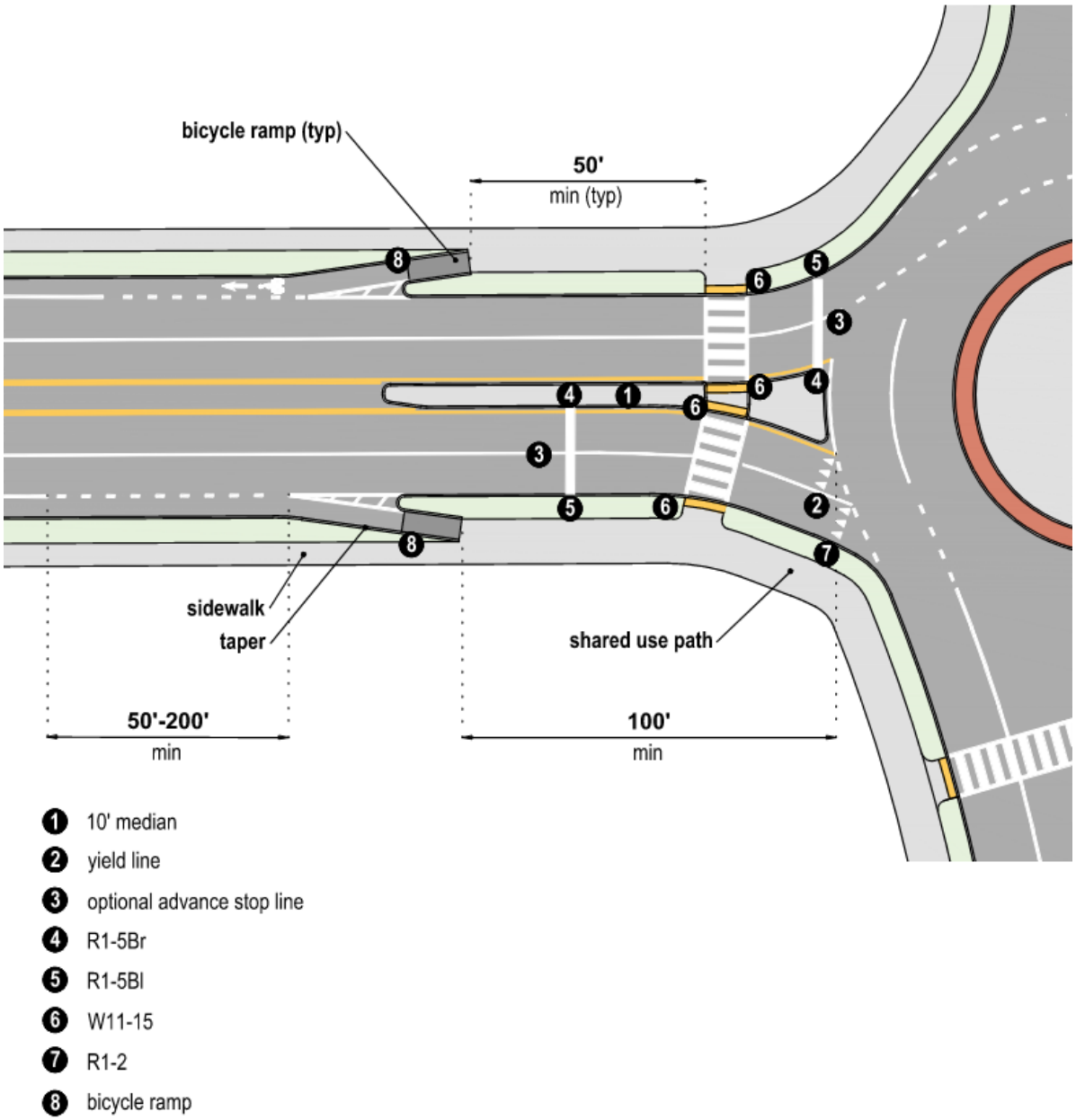


Figure 18-55: Typical Layout of Bike Lane Transitions to Shared Use Path at Multilane Roundabout with Bike Ramps

(Refer to the **TMUTCD** and **Section 18.1.1** 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.

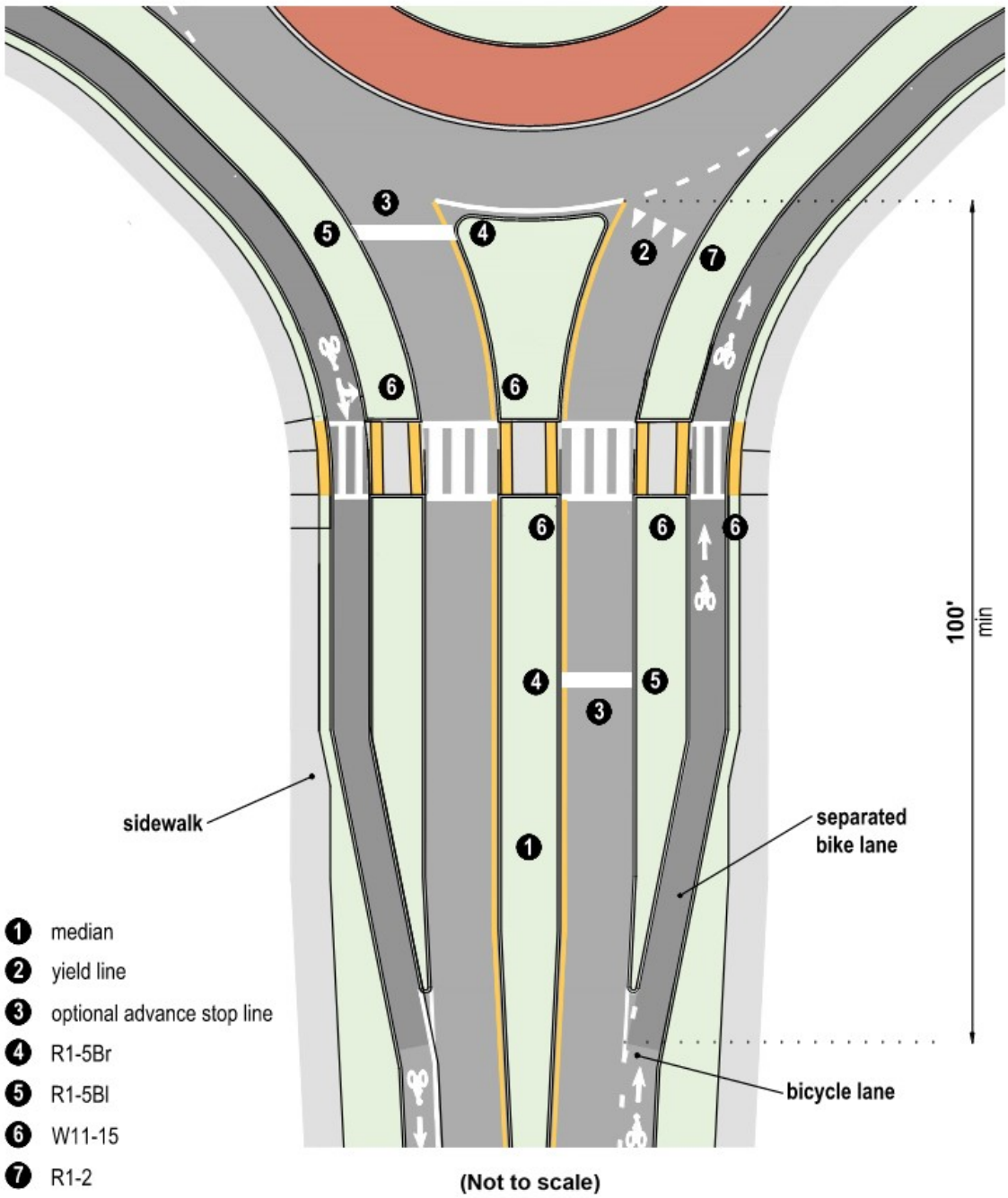


Figure 18-56: Typical Layout of Separated Bike Lanes at Roundabout

(Refer to the **TMUTCD** and **Section 18.1.1** 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.

At multilane roundabouts at least one of the following additional treatments to alert motorists to the presence of pedestrians or to require them to slow or stop traffic may be required to accommodate people with disabilities at the crossing:

- A raised crosswalk;
- An actuated rapid flashing beacon;
- A pedestrian hybrid beacon; and
- A traffic control signal with pedestrian signal heads.

18.5.7 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.

18.5.7.1 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 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 18-57** shows the design detail of a bikeway crossing parallel tracks.

18.5.7.2 Crossing Tracks

Railroad tracks that cross a street typically cross at an angle near 90-degrees. **Figure 18-57** shows an example treatment for a bikeway crossing railroad tracks at a skew.

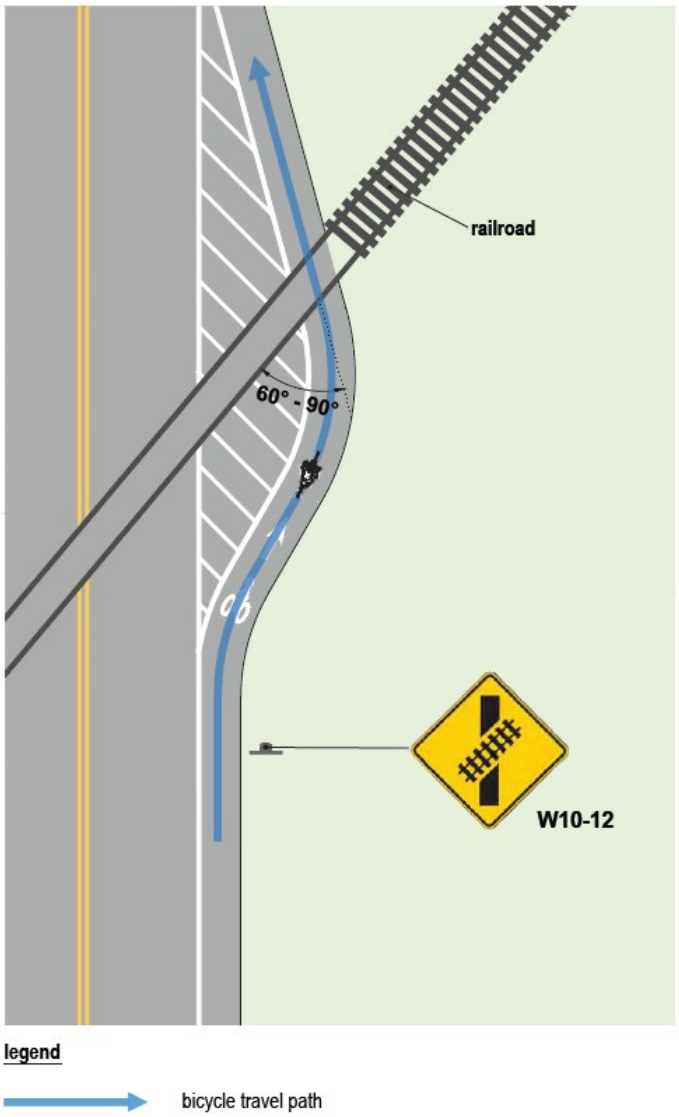


Figure 18-57: Railroad Crossing Example

(Refer to the **TMUTCD** and **Section 18.1.1** for additional signing and pavement marking guidance.)

18.6 Maintenance, Operations, and Work Zone

18.6.1 Maintenance Considerations

As discussed in **Section 4.13**, a roadway can and should be located and designed to complement its environment and serve as a catalyst to environmental improvement. Projects which add bicycle facilities should be evaluated for opportunities to achieve these goals.

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. E-Scooter riders have higher rates of injuries compared to bicyclists due to falls associated with pavement defects such as potholes or cracks, or from hitting sign posts on sidewalks or curbs.

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 and clean bikeway to minimize or eliminate the need for hand-sweeping to clear bike lane debris. On separated bike lanes this can be a challenge where the width is narrow. 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 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.

18.6.2 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. In situations where higher volumes of bicyclists will have to share the lane with traffic, and those bicyclists are likely to include children, consideration should be given to providing a flagger to assist in the management of traffic flow. See **Chapter 23** for additional Temporary Traffic Control Guidance.

Chapter 19
Pedestrian Facilities

19.1 Overview

This chapter discusses bicycle facilities and includes the following sections and subsections:

19.1 Overview
- 19.1.1 Purpose;
- 19.1.2 Design Imperative - Safety;
- 19.1.3 Relationship to Other Policies, Laws, and Regulations;
- 19.1.4 Definitions;
- 19.1.5 Provision of Pedestrian Facilities;
- 19.1.6 Project Development; and
- 19.1.7 Maintenance and Operation.

19.2 Elements of Design
- 19.2.1 Design User; and
- 19.2.2 Accessibility Requirements.

19.3 Linear Pedestrian Facilities
- 19.3.1 Characteristics Contributing to Pedestrian Comfort and Safety Along the Roadway;
- 19.3.2 Sidewalk Design on Curbed and Non-Curbed Roadways; and
- 19.3.3 Shared Use Paths.

19.4 Curb Ramp Design
- 19.4.1 Curb Ramp Locations;
- 19.4.2 Design Considerations;
- 19.4.3 Curb Ramp Components;
- 19.4.4 Design Elements;
- 19.4.5 Curb Ramp Types;
- 19.4.6 Curb Ramp Evaluation; and
- 19.4.7 Design Variance to Curb Ramp Replacement.

19.5 Driveway Design Considerations
- 19.5.1 General Guidelines; and
- 19.5.2 Designing for Cross Slope Compliance.

19.6 Intersections and Crossings
- 19.6.1 Pedestrian Crossing Principles;
- 19.6.2 Intersection Crossing Treatment Decision-Making Framework;
- 19.6.3 Geometric Treatments for Pedestrians at Intersections;
- 19.6.4 Uncontrolled Crossing Safety Countermeasures;
- 19.6.5 Signalized Crossing Safety Countermeasures; and
- 19.6.6 Mid-block Pedestrian Crossing Safety Countermeasures.

19.1.1 Purpose

The purpose of this Pedestrian Facilities chapter 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 19.2.1**.

19.1.2 Design Imperative - Safety

Safety is a key factor in planning and design of roadway facilities for pedestrians as they are particularly vulnerable street users. **43 TAC §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 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 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 19-1**). Note that **Figure 19-1** 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. Designers should take into account the number and severity of pedestrian conflict points at a given intersection. 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 applying pedestrian guidance in this manual, 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.

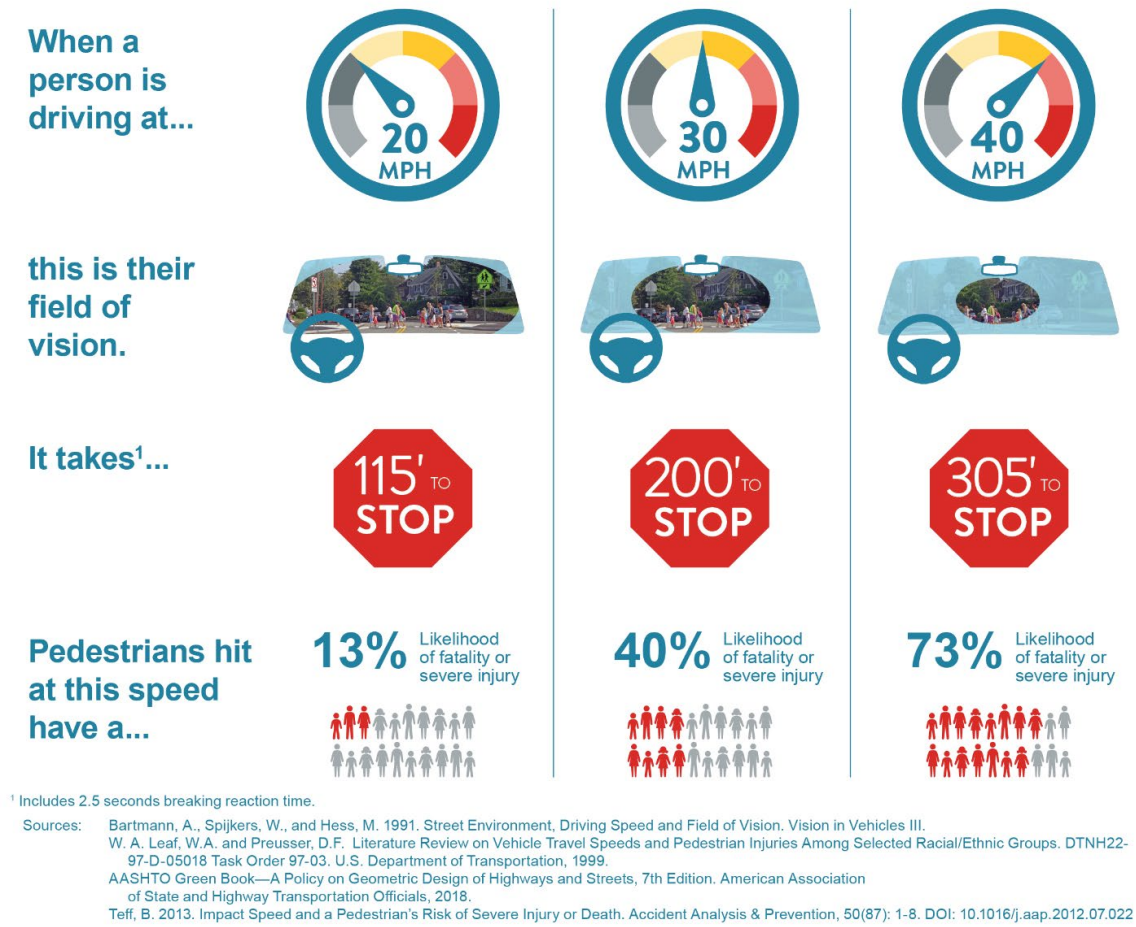


Figure 19-1: Speeds and the Risk of Serious Injury to Pedestrians.

19.1.3 Relationship to Other Policies, Laws, and Regulations

This design chapter is intended to support and comply with existing State and federal guidance. Under **23 USC 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 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 **43 TAC §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. **43 TAC §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, **43 TAC §15.122(2)** states: "The factors provided in paragraph (1) of this section will be assessed when developing transportation projects. **43 TAC §15.122(2) (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."

19.1.3.1 Accessibility

Where pedestrian use is permitted, roadway designs and alterations must comply with accessibility requirements established by the **ADA Standards (2010)** adopted by the U.S. Department of Justice (DOJ), the **ADA Standards (2006)** adopted by the USDOT, **PROWAG**, and **TAS**.

While the **ADAAG** are the current enforceable federal regulations implementing the **Americans with Disabilities Act of 1990 (ADA)**, these pertain mostly to accessible design at public facilities and their grounds. Since the **ADAAG** does not adequately address features unique to public ROW, on August 8, 2023, the US Access Board published the **PROWAG** to provide guidance. The **PROWAG** has not been formally adopted by the DOJ or the DOT so it is not yet enforceable. However, **PROWAG** provides a useful framework to meet TxDOT's obligations to make our programs, services, and activities in the public ROW

readily accessible and useable by all individuals including those with disabilities.

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 but will require revisions to align with the guidance included in **PROWAG**.

As of May 15, 2017, changes to the **16 TAC Part 4, §68.102** allow the TDLR to accept compliance with **PROWAG** in lieu of **TAS** for projects in the public ROW. Because the FHWA encourages the use of **PROWAG** as best practice, TxDOT designers must use **PROWAG** to achieve accessible design requirements in the public ROW. **TAS** and DOJ **ADA Standards (2010)** must be used for design and construction of buildings such as TxDOT buildings. However, sites in the public ROW, such as safety rest areas, will use **PROWAG** for applicable guidance. 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 chapter, **PROWAG**, 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 details on the design implications of compliance are provided in **Section 19.2.2**.

19.1.3.2 Key Pedestrian Laws and Definitions

The **TTC, Title 6**, defines laws associated with pedestrian facilities and pedestrian ROW (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.

19.1.3.2.1 Marked and Unmarked Crosswalks

Texas law (**TTC §541.302**) 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 19-2** 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.

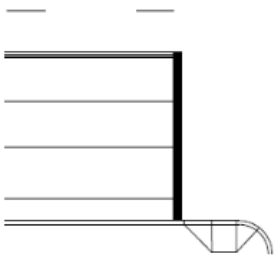


Figure 19-2: An Unmarked Crosswalk Exists at an Intersection as the Extension of the Sidewalk or Edge of Roadway Across the Intersecting Street, Regardless of the Presence of a Pedestrian Access Route.

19.1.3.2.1.1 Right of Way at Street Crossings

Motorists must stop and yield the ROW to pedestrians who are lawfully within crosswalks at:

- All marked or unmarked unsignalized intersection or midblock crossings unless pedestrians are legally prohibited (**TTC §552.003**); and
- 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 (**TTC §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 ROW to motorists at all unsignalized unmarked midblock locations and between adjacent intersections at which signals are in operation. A pedestrian may cross only in a marked crosswalk (**TTC §552.005**).

19.1.3.2.1.2 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 ROW 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, grade, etc.), shoulders generally should not be used as pedestrian accessible routes. Other options should be pursued when designing pedestrian accessible routes.

19.1.4 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 ROW. A PAR is contained within the pedestrian circulation path.

- **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 ROW components. Refer to the **PROWAG** webpage for more information.

19.1.5 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. Designers should conform to approved pedestrian or active transportation plans affecting project area. See **TxDOT Active Transportation Plan** for more information.

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 ROW, 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.

For alteration projects, existing physical constraints sometimes make it technically infeasible to meet certain accessibility requirements. Existing physical constraints include, but are not limited to, underlying terrain, underground structures, adjacent development facilities, drainage, or the presence of significant natural or historical features; however, available ROW is not considered an existing physical constraint. In these situations, compliance is required to the maximum extent feasible in all other elements of design. Any non-compliant conditions must be documented by submitting a variance to TDLR with justification for non-compliance. Non-compliant elements must be reported to the District ADA liaison and included in the **TxDOT ADA Transition Plan**. For additional information on the **TxDOT ADA Transition Plan** contact the Landscape Architecture Branch of the DES. For new construction projects (e.g. in greenfield conditions), all design elements must be fully compliant with **PROWAG**.

19.1.5.1 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 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;
    - 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 typically considered or designed to be a PAR; however, a shoulder can be designed to function as a PAR where it meets accessibility requirements for cross-slope and width in areas where that is the most practical method to accommodate pedestrians until such time as a sidewalk or shared use path is constructed.

19.1.6 Project Development

Planning for pedestrian and multimodal facilities must occur early and continuously throughout project development and must follow TXDOT's **PDP**. 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;
- On-street parking;
- 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;
- Should be upgraded or maintained as part of the project; and/or
- Altered by the original scope of the project (per the FHWA/DOJ joint technical memo defining alterations and maintenance activities).

The evaluation requires further detail for the following elements:

Sidewalks. Suitability of width to context, while adhering to minimum width standards per accessibility guidelines (volume of expected users, see **Section 19.3.2**).

Suitability of placement to context (characteristics of adjacent roadway, see **Section 19.3.1**).

Determination of the feasibility of filling gaps between existing ramps and nearby sidewalk segments that are disconnected from the street corner.

Curb ramps. Provision of curb ramps during resurfacing projects.

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.

19.1.7 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 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.

19.2 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 19.3.1** and **19.6.1**.

19.2.1 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.

19.2.1.1 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.

19.2.1.2 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 19-3**. 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 19.2.2** for information on accessibility requirements.

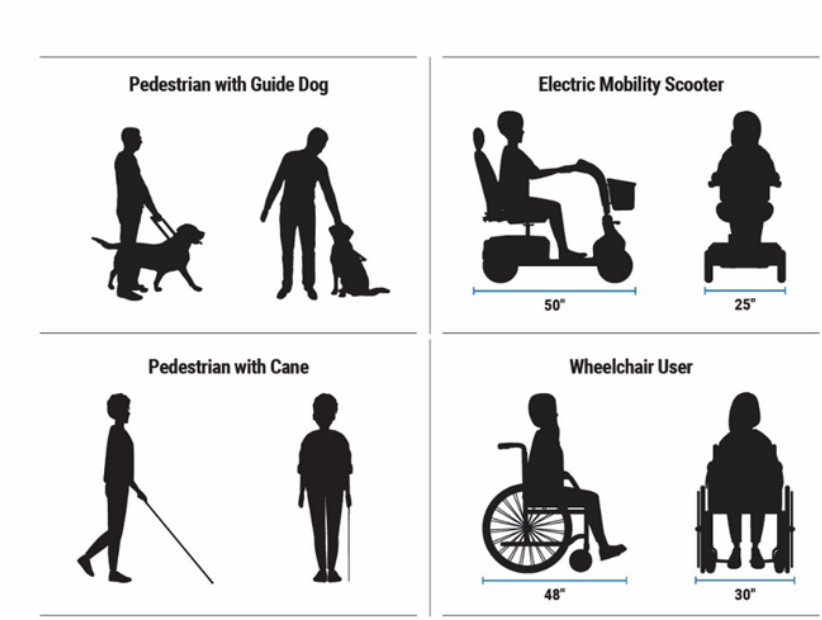


Figure 19-3: Mobility and Assistive Devices and Users.

19.2.1.3 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 or less used where older pedestrians are expected or near schools. Refer to the **HCM** for criteria for slower walking speed consideration. Refer also to the **TMUTCD** Part 4 Section 4.E06, and **PROWAG**, with regard to pedestrian signal timing.

19.2.2 Accessibility Requirements

As described in **Section 19.1.3.1** above, projects affecting pedestrian facilities must comply with accessibility requirements established in the **PROWAG**.

19.2.2.1 Pedestrian Access Routes (PAR)

A 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 ROW, 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; and
- Pedestrian overpasses and underpasses, and similar structures, including any elevators used for their access;

- Elevators and ramps; and Gates.

Figure 19-4 shows the protected zone in plan and cross section view.

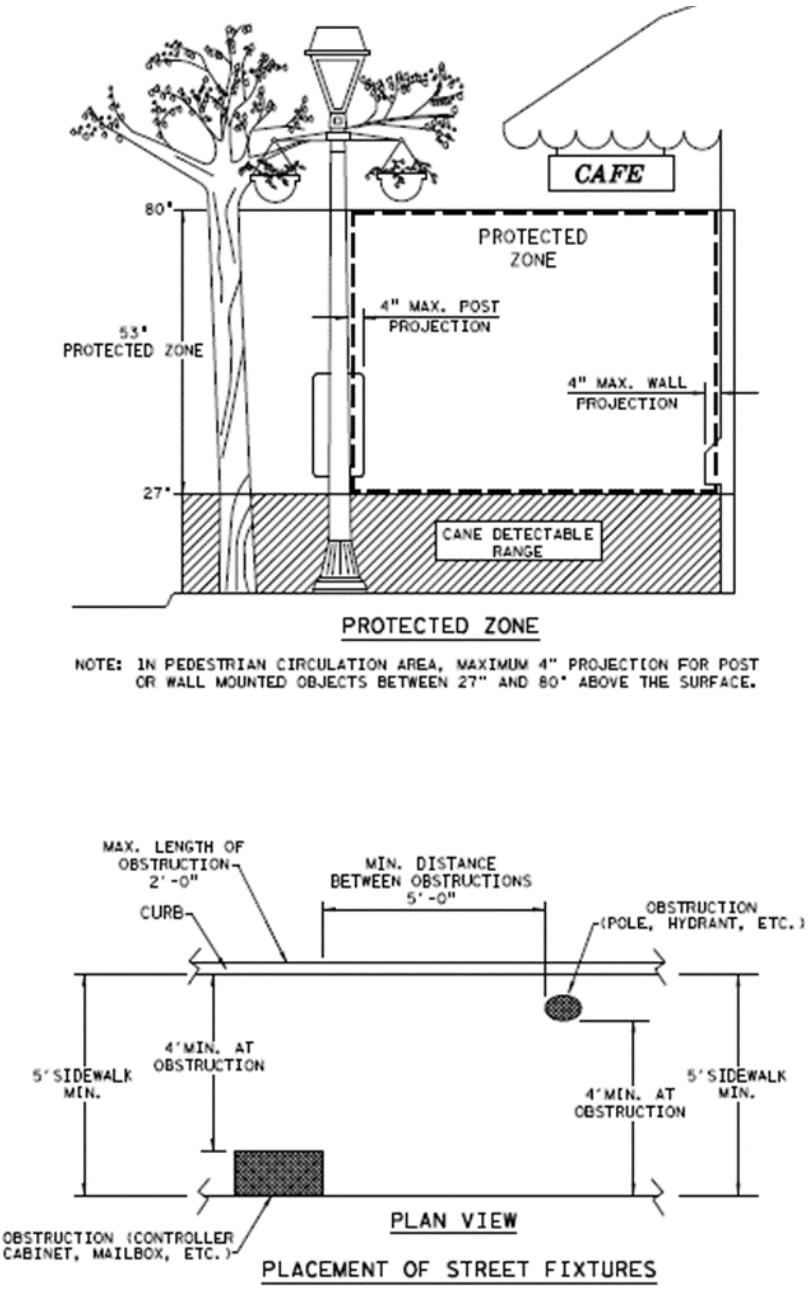


Figure 19-4: The Protected Zone (Top) and The Plan View (Bottom) Regarding Obstacles and Protruding Objects.

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.

19.2.2.2 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 18**.

19.2.2.3 Obstacles and Protruding Objects

Obstructions are objects along or overhanging any portion of the sidewalk or other pedestrian circulation path. 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. **Figure 19-5** depicts barriers for vertical clearance under 80-in. 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 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, a minimum clearance of 8-ft for overhead obstructions is recommended to accommodate people riding bicycles and operating Micromobility devices, including on pedestrian bridges which may be used by bicyclists and Micromobility users.

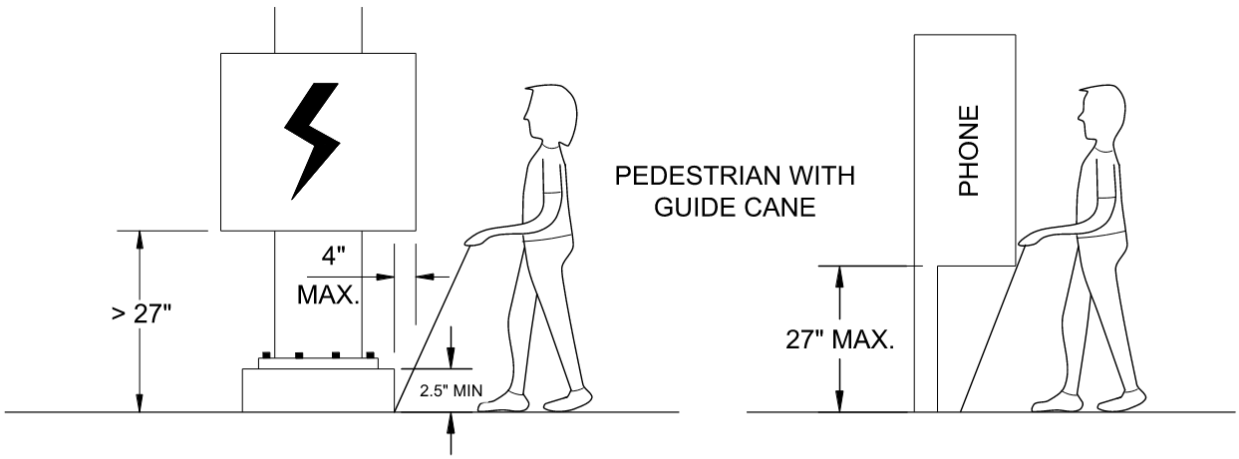


Figure 19-5: Detection Barriers for Vertical Clearance under 80".

19.2.2.4 Surface, Grade and Cross Slopes

19.2.2.4.1 Surface

The surface material of the PAR must be firm, 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 are prohibited at grade breaks (such as at the top of curb ramps). Vertical discontinuities of ¼ inch maximum are permitted to be vertical. Between ¼ inch and ½ inch, vertical discontinuities must be beveled at a maximum 2:1 slope. Vertical discontinuities greater than ½ inch and up to 6-in must be beveled at a maximum 8.3% slope. Changes in level greater than 6-in must be designed as a ramp.

Considerations for surfacing of shared use paths are discussed in **Chapter 18**.

19.2.2.4.2 Detectable Warning Surfaces

Detectable warnings indicate to pedestrians, especially pedestrians with vision disabilities, 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 cues for the pedestrians. Section R305 of **PROWAG**, **Section 19.4.4.5** of this document, and the current **TxDOT Pedestrian Facility Standards**

detail the application of detectable warnings in the public ROW.

19.2.2.4.3 Maximum PAR Grade (Within the Street or Highway ROW)

The following three conditions have varying maximum PAR grade requirements:

- 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 on an independent alignment: For a PAR facility with an independent alignment, the grade of a PAR must not exceed 5 percent.
- Pedestrian Crossings of roadways: The running slope (measured parallel to direction of pedestrian travel) of a PAR has a maximum grade of 5 percent. Where roadways are superelevated, the grade of the running slope of the crossing may be the same as the superelevation rate.

Where compliance with the above is not technically feasible due to existing terrain or infrastructure, a notable natural feature, or similar existing physical constraints, a variance application must be submitted, and compliance is required to the maximum extent feasible.

19.2.2.4.4 Cross Slope

The cross slope of a PAR must not exceed 2.0 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 2.0 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 or yield control (stop sign or yield sign) must have a maximum 2.0 percent cross slope. Signalized intersections, or those without stop control or yield control, must not exceed 5 percent cross slope. Where the PAR is contained within midblock pedestrian street crossings or at a roundabout, the cross slope shall not exceed the street or highway grade.

19.2.2.5 Accessible Pedestrian Signals

Section R206 of **PROWAG** specifies that where pedestrian signal heads are provided at street crossings, the crossing must include accessible pedestrian pushbuttons complying with R307 of **PROWAG**. This applies to new projects or those where the signal controller and software are altered, or the signal head is added or replaced. See **Section 19.6.5** of this document for guidance and TRF Division Guidance.

19.2.2.6 Micromobility Device Accessibility Considerations

As discussed in **Chapter 18,** micromobility devices include small, fully or partially human-powered vehicles such as electric scooters, motorized wheelchairs, and other power-driven mobility devices as well as electric-bikes (e-bikes), e-scooters, and other similar devices. Many of these devices, such as motorized wheelchairs and electrical bicycles, may serve as a mobility device for pedestrians with disabilities.

They are increasingly common in urban areas and suburban areas in Texas but may also be found in some rural areas. These devices may either be privately owned or part of fleets that are publicly available for short-term rentals. Fleet devices are most often parked in the ROW 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 devices, it is recommended to identify adequate width outside of the PAR for parking of the micromobility devices. This parking area may be in the buffer zone of a 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 these device parking areas. The boundaries of the parking areas should be clearly identified for the use of micromobility devices 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 devices outside the PAR.

In areas serviced by docked micromobility devices, proposed locations for new docking stations must be coordinated with the local government according to its requirements for micromobility devices. 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.

19.3 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 travel lanes. As such, the desirable placement of the linear pedestrian facility is as close to the ROW line as practical.

19.3.1 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.

19.3.2 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.

19.3.2.1 Pedestrian Zone

The pedestrian zone, which is also known as the “walking zone” or pedestrian circulation path, incorporates the PAR. The PAR is the portion of the pedestrian zone dedicated to ADA accessible pedestrian movement which must meet specific grading requirements and be free from obstructions or protruding objects.

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 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 2.0 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, though more frequent passing areas should be prioritized.

While sidewalks do not need to be perfectly straight, the pedestrian zone should not weave back and forth in the ROW for no other reason than to introduce curves. Meandering sidewalks create navigational difficulties for pedestrian with vision disabilities.

19.3.2.1.1 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. A minimum of 8-ft of width is needed at all transit stops to accommodate boarding and alighting. 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.

19.3.2.1.2 Uncurbed Roadways

The same width requirements for the pedestrian zone apply to sidewalks on uncurbed roadways. In these cases, sidewalk should typically be placed between the ditch and the ROW line, not between the ditch and edge of the vehicular travelway. See **Figure 19-6**.

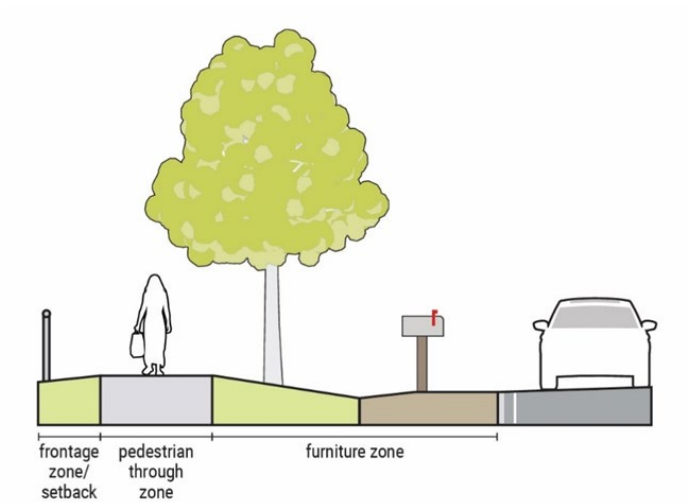


Figure 19-6: An Uncurbed Roadway.

19.3.2.2 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.

19.3.2.2.1 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 ROW, including sign posts, bus stops, parking meters, utility connections, light poles, and other appurtenances. 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. Refer to TXDOT's **Landscape and 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 19.5.2** 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 lower-speed curbed streets. See **Table 4-20** 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 wider 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 4-20** would be the desirable buffer width.

19.3.2.2.2 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 4-20** in the RDM.

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.

19.3.2.3 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.

19.3.2.3.1 Curbed Roadways

Frontage zone width may vary between 1-ft 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.

19.3.2.3.2 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.

19.3.3 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 ROW (commonly referred to as sidepaths) or within an independent ROW.

Shared use paths are used by pedestrians and must meet the accessibility requirements of the ADA and be designed to conform to the **PROWAG**. Where pedestrian and bicycle travel are accommodated on separate pathways, only the pedestrian pathway must meet the requirements of the above guidelines. See **Chapter 18, Section 18.4.1** for more information on shared use paths.

19.3.3.1 Cross Slope

See **Section 19.2.2.4.4** for information regarding the accessibility requirements related to cross slope.

19.3.3.2 Grade

See **Section 19.2.2.4.3** for information regarding the accessibility requirements related to grade. Shared use paths (on an independent alignment from the roadway) may only exceed 5 percent maximum grade at overpasses and underpasses, where the grade may be designed with a series of ramps and landings. See **Section 19.7.3**.

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. At locations with sustained lengths of grades of 5 percent or more, level landings within the shared use path can be provided to provide periodic resting spaces for pedestrians. These should be designed to:

- Permit users to stop and periodically rest within a level area (graded to 2% or less in all directions);
- Be as wide as the shared use path, with a landing length of at least 6 feet;
- Provide smooth grade breaks using vertical curve design criteria for shared use paths; and
- At locations where stopped users within the path are likely to create conflicts with other users of the path, level-graded pull-outs should be considered.

Spacing of level landings should consider the sustained length of the grade and the steepness of the grade. Engineering judgement will be required to determine landing spacing, but consideration should be given to travel time between landings with landings located at evenly spaced intervals.

Alternately or in addition to a lower sustained grade, level-graded pull-outs (or rest areas) 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. Where level pull outs are provided, they should be designed to:

- Permit users to stop and periodically rest within a level area (graded to 2% or less in all directions) outside the path of travel;
- Avoid abrupt grade breaks with smooth vertical curves at transition meeting shared use path vertical curve design guidance;
- Be a minimum of 6 feet by 6 feet in dimension; and
- Consideration should be given to providing seating within this space that meet ADA requirements.

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 19.12** for guidance regarding protective rails adjacent to steep slopes.

Provision of a handrail may impact shared use path width. See **Chapter 18** for information regarding the impact of handrails on path width.

19.3.3.3 Additional Requirements

More information on shared use paths can also be found in **Chapter 18**.

19.3.4 Linear Pedestrian Facility Design Criteria

Table 19-1 shows design criteria for linear pedestrian facilities. These design values may be adjusted based on the performance based practical design evaluation as discussed in **Chapter 2**. The previous and subsequent subsections in this chapter provide detailed discussions for each design element and factors that can influence these values.

Table 19-1 Design Values for Linear Pedestrian Facilities

	Design Element	Minimum or Maximum	Desirable	Reference
Pedestrian Access Route (PAR) (e.g. Sidewalk)	Clear Width1	5-ft	6-ft to 8-ft	See 19.2.2.2 and 19.3.2.1
	Buffer Zone Width (Curbed Roadways)	4-ft from FOC	6-ft or more from FOC	See 19.3.2.2.1
	Buffer Zone Width (Uncurbed Roadways)	10-ft from Edge of Pavement	Desirable Clear Zone Width	See 19.3.2.2.2
	Frontage Zone Width (Curbed Roadways)	0-ft	1-ft or more	See 19.3.2.3.1
	Frontage Zone Width (Uncurbed Roadways)	See 19.3.2.3.2		
	Maximum Cross Slope	2%	1.5%	See 19.2.2.4.4
	Maximum Pedestrian Crossing Cross Slope at Intersections	5%	2% or less	See 19.2.2.4.3 and 19.6
	Maximum Longitudinal Grade (Adjacent to Roadway / Curb)	Must not exceed general grade of the roadway		See 19.2.2.4.3
	Maximum Longitudinal Grade (Independent Alignment)	5%		See 19.2.2.4.3
	Obstacles and Protruding Objects	See 19.2.2.3		
Shared Use Path (SUP)	See 18.3 and 18.4.1			
Note	1. 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. A design variance will be required if this criteria is not met.			

19.4 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 midblock crossings. The TxDOT preferred standard is two curb ramps per corner, each aligning with desired paths of travel (see **Section 19.5.2.6**). **PROWAG** requires two curb ramps or a blended transition. Note that a single diagonal ramp is only permitted if two ramps or a blended transition are technically infeasible. All curb ramps must be compliant with **PROWAG**. Refer to the current **TxDOT Pedestrian Facility Standards** for further detail on curb ramp design.

19.4.1 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 and accessible push buttons*;
- There is a marked or unmarked crosswalk; or
- There is a school crossing.

*The ADA requires installation of curb ramps even in the absence of a pedestrian walkway to allow pedestrians to reach accessible push buttons.

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 pedestrians to detect and maneuver. See **Section 19.1** for the design of curb ramps at driveways.

19.4.2 Design Considerations

Manhole covers, grates, and obstructions should not be located within the curb ramp, maneuvering area, landings or turning spaces. Where they must be located in the ramp or landing, they must be firm, stable, and slip resistant. Curb ramps should not be located where pedestrians must cross drainage structures such as inlets and manholes. Where a drainage grate must be located in the path of pedestrian travel, it must be ADA compliant with openings that prohibit passage of a sphere larger than ½ inch. In new construction, curb ramp design must consider both the location of the curb ramp and the drainage and utility 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 bottom of the ramp. Curb ramps

must be located away from low points of the curb return. For more information on drainage design, see **Section 4.12**.

19.4.3 Curb Ramp Components

The basic components to the standard curb ramp design are the following and are shown in **Figure 19-7** (see **Section 19.4.4** 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 curb ramps when a change in direction is necessary to access the ramp.
- **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 side of a curb ramp, or when the curb ramp is adjacent to a walkable surface.
- **Gutter and Clear Area** – The gutter is the curb and gutter section within the ramp width that connects the roadway travelway to the ramp. The Clear area is space provided beyond the bottom grade break of the curb ramp, which must be located wholly outside the vehicle travel lanes, including bike lanes, that run parallel to the crosswalk.

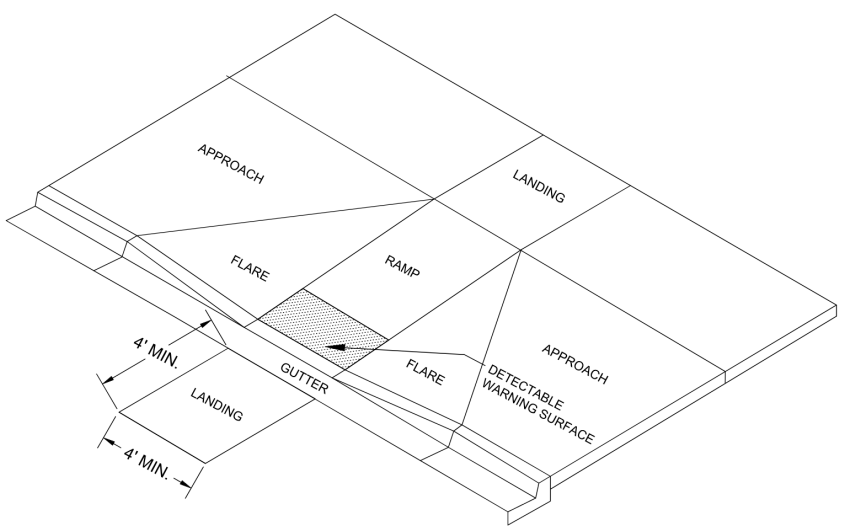


Figure 19-7: Curb Ramp Components.

19.4.4 Design Elements

19.4.4.1 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).

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 13.3 percent. If the algebraic difference in grade exceeds 13.3 percent, a 24-in level strip (maximum 2 percent slope) must be provided to transition from the curb ramp to the gutter (or road surface) to avoid tripping concerns (see **Figure 19-8**).

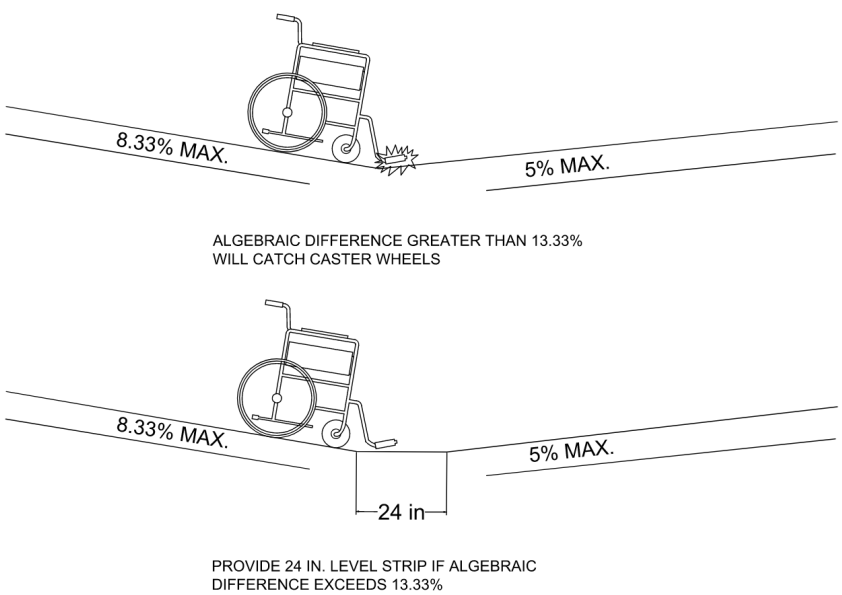


Figure 19-8: Grade Considerations for Curb Ramps.

19.4.4.2 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 2.0 percent at stop and yield controlled intersections, and 5 percent at signalized and uncontrolled intersections. At midblock pedestrian street crossings, **PROWAG** will permit curb ramp cross slopes to be equal to or less than the street or highway grade. It is desirable to closely match the street grade because introducing warping of the cross slopes on the ramp can cause people with mobility disabilities to lose contact with one of their wheels on the ramp and cause instability.

19.4.4.3 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 or blended transition connecting to a sidewalk is 4-ft, exclusive of flared sides. The minimum width of a curb ramp or blended

transition connecting to a shared use path is at least as wide as the shared use path. Refer to the latest **TxDOT Pedestrian Facility Standards** for ramp types.

19.4.4.4 Turning Spaces (Landings)

A turning space is a landing area that allows users to maneuver and make turns to travel 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 2.0 percent in the direction perpendicular to the curbline and less than or equal to the grade of the roadway perpendicular to the curb ramp. Where a turning space serves two curb ramps, the slopes must be less than or equal to the parallel roadway grades. 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.

19.4.4.5 Clear Area (Gutter)

At the bottom of curb ramps, the space within the street (at the gutter) serves as a clear area to traverse to and from the curb ramp. The minimum dimensions for this clear area are 4-ft x 4-ft, and the clear area must be located wholly outside of travel lanes (including bicycle lanes) that run parallel to the crosswalk. The slopes of the clear area are governed by the crosswalk cross slopes as discussed in 19.4.4.2.

19.4.4.6 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 for rail vehicles where the edges of the boarding platform are not protected by screens or guards;
- Boarding platforms for buses where the curb height is higher than a standard height curb; and
- Where the side of the boarding and alighting areas facing the rail vehicles is not protected by screens or guards.

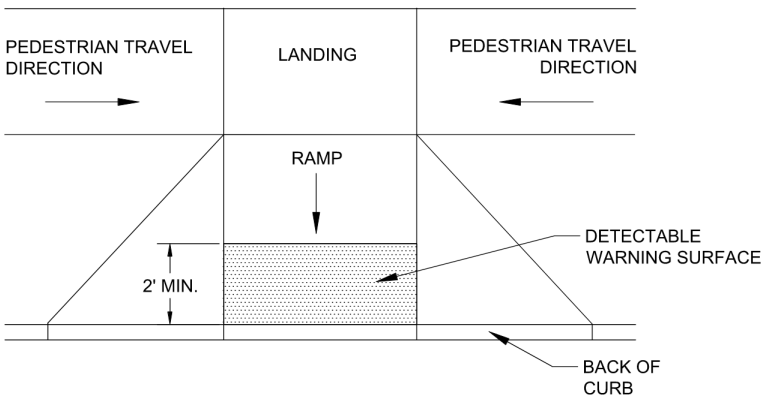


Figure 19-9: Detectable Warning Strips with a Minimum of 2-ft.

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; however, where a concrete border is required for proper installation of a detectable warning surface, a concrete border not exceeding 2 inches shall be permitted on all sides of the detectable warning surface except between the detectable warning surface and the edge of pavement where a setback is already permitted. Detectable warning surfaces are generally placed at the back of the curb or edge of street and they may be curved along the corner radius. For directional curb ramps the detectable warning surfaces must be placed at the bottom of the grade break except where one edge of the detectable warning surface would be greater than 5-ft from the back of curb.

Detectable warning materials must meet current **TxDOT Pedestrian Facility Standards** materials specifications.

19.4.4.7 Side Treatments

Side treatments of curb ramps include flares and returned curbs. See images below for an example of flares and returned curbs. **Figure 19-10** depicts the different types of side treatments.

19.4.4.7.1 Flares

Flares must be provided for curb ramp designs where the pedestrian circulation path crosses the curb ramp. Flares are not intended to be part of the pedestrian access route. Flared sides must be sloped no more than 10 percent measured parallel to the curb.

19.4.4.7.2 Returned Curbs

Returned curbs do not use flares, but instead use curbs on the sides of the curb ramp. Returned curbs may only be used where a pedestrian circulation path does not exist along the edge of ramp, either because the adjacent surface is planted, substantially obstructed, or otherwise not meant for pedestrian travel.



Figure 19-10: Curb Ramp with Flares (left) and Returned Curb (right).

19.4.5 Curb Ramp Types

Curb ramps are classified as perpendicular, directional, parallel, combination or blended transition per **TxDOT Pedestrian Facility 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 make it technically infeasible to construct two curb ramps or a blended transition. Such ramps are typically not aligned with either of the crosswalks that extend across the intersecting streets. These ramps may direct a pedestrian with vision disabilities directly toward the center of the intersection.

19.4.5.1 Perpendicular

Perpendicular curb ramps are designed to be perpendicular to the curbline and are the preferred ramps for application over parallel and directional curb ramps. **Figure 19-11** depicts a perpendicular curb ramp.

Where practical, the curb ramp path should be aligned with the crosswalk; however, 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, and in these cases a directional curb ramp should be considered (see **Section 19.4.5.3**).

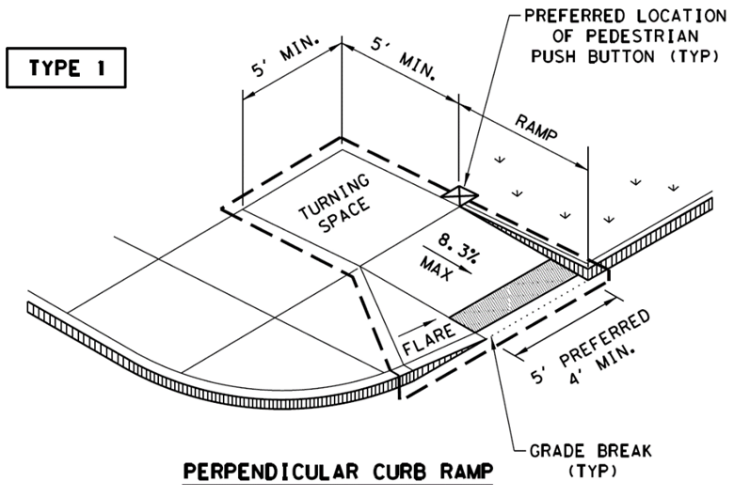


Figure 19-11: Specifications for a Perpendicular Curb Ramp.

19.4.5.2 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 a 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 19-12** shows an example of a parallel curb ramp.



Figure 19-12: Parallel Curb Ramp with Curb Wall.

19.4.5.3 Directional

Directional curb ramps are designed to align with the pedestrian crosswalk direction. Where a perpendicular ramp alignment would exceed a 20-degree angle with the direction of the crosswalk, a directional curb ramp should be considered. Directional curb ramps are

designed like perpendicular ramps; however, instead of aligning perpendicular to the curbline they align parallel with the crosswalk. Directional curb ramps require the curb ramp grade breaks to be perpendicular to the direction of pedestrian travel, and result in a triangular clear area at the bottom of the grade break which must not exceed 2.0% in the direction of pedestrian travel.

19.4.5.4 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 with physical constraints 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 19-13** depicts the components of a combination curb ramp.

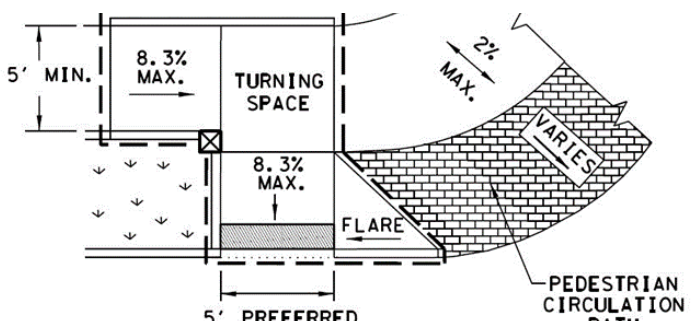


Figure 19-13: Combination Curb Ramp. Key Ramp Slopes and Turning Space are Maintained.

19.4.5.4.1 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 to allow people traveling on the sidewalk to bypass the blended transition. Otherwise, the maximum slope is 2.0 percent. Blended transitions without accessible pedestrian signals (APS) should be used sparingly since they do not provide directionality for pedestrians with vision disabilities. **Figure 19-14** shows an example of a blended transition curb ramp.



Figure 19-14: A Blended Transition Curb Ramp.

19.4.5.4.2 Pedestrian Refuge at Median and Channelization Islands

Pedestrian refuge islands typically provide a level cut-through at medians, splitter islands, and channelizing islands. The minimum clear width of the pedestrian refuge is 5-ft or as wide as the shared use path if connecting to a path. The width of the median must be at least 6-ft or greater to serve as a pedestrian refuge. Tactile detectable warnings must be placed no more than 6-inches from the edge of the travel lane (or at the back of curb) and the two detectable warning panels must be separated by at least 2-ft; as such, if the median width is only 6-ft wide the detectable warning surfaces must be placed at the face of curb to provide the 2-ft of separation. For medians less than 6-ft, a pedestrian refuge is not provided and detectable warnings shall not be utilized. **Figure 19-15** shows a level cut through with median refuge. **Figure 19-16** provides an option if a wider median area is available.

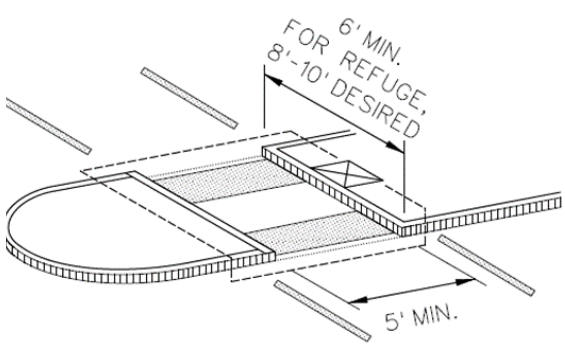


Figure 19-15: Curb Ramp with Level Cut-through at Median.

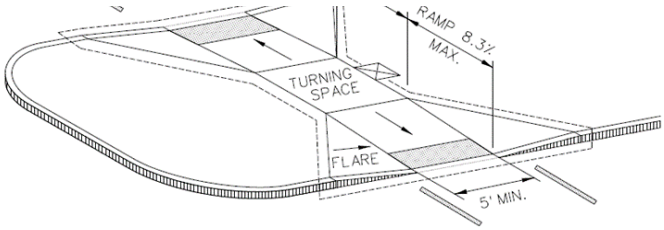


Figure 19-16: Curb Ramps at a Median (Option for wider medians).

19.4.6 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;
- 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.

19.4.7 Design Variance to Curb Ramp Replacement

Where existing physical constraints make it technically infeasible for altered elements, spaces, or facilities to fully comply with the requirements of **PROWAG**, compliance with the

ADA is required to the maximum extent feasible within the scope of the project. A variance application must be submitted for any element that does not achieve full compliance.

19.5 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.

See Chapter 16 for more detailed driveway design guidance.

19.5.1 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 **Chapter 16**). 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 should not be provided where minor driveways cross a sidewalk. However, if the driveway is controlled with a traffic signal or a stop sign, detectable warnings are required.
- Where separator islands are provided between inbound and outbound travel lanes in a driveway, the island must provide a pedestrian refuge with a minimum width of 6-ft. See **Section 19.4.5.4.2** for the design of pedestrian refuge islands. **Figure 19-17** depicts a separator island.
- Driveways must maintain the PAR as a continuous, ADA compliant grade, 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 19.2.2**).
- Locate sidewalks far enough from the curb, or edge of pavement (4-ft or greater is preferred), to provide a suitable 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 19-18**). Vehicles should not block the sidewalk when parked in driveway.

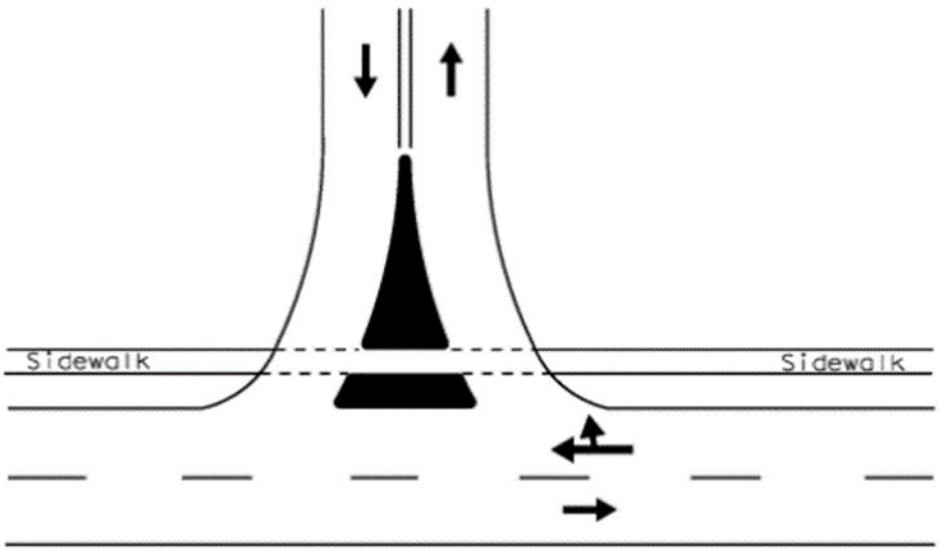
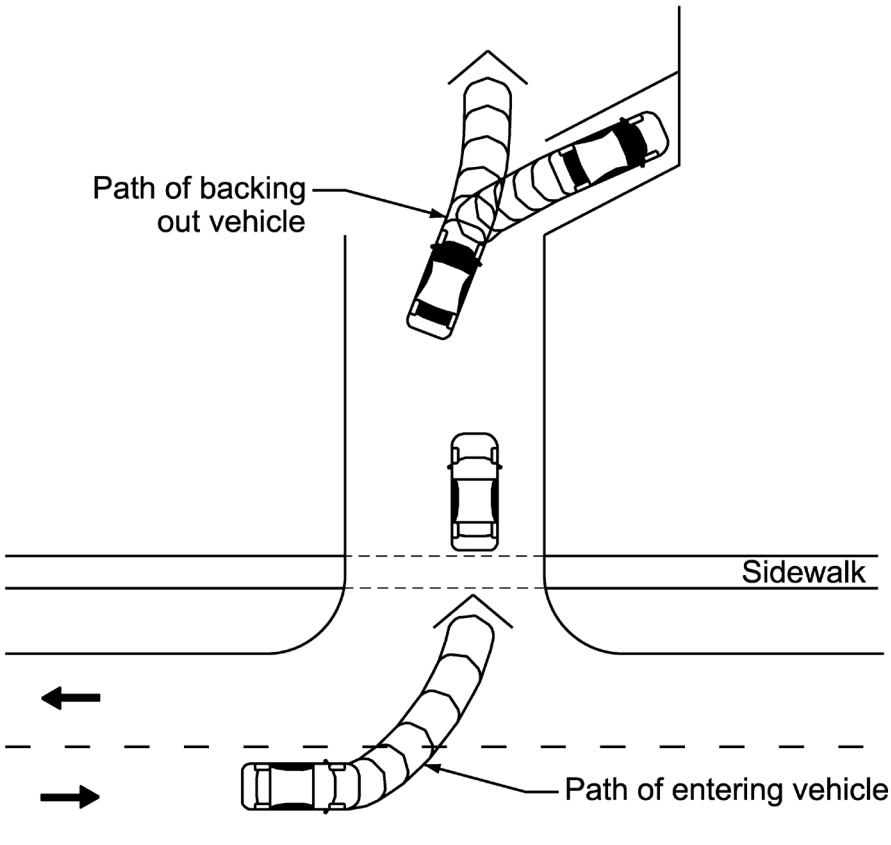


Figure 19-17: Channelizing Island Provide Pedestrian Refuge.



A. The driver of an entering vehicle stops after crossing the sidewalk, then waits for a vehicle backing out to clear the driveway throat.

Figure 19-18: Sufficient Throat Length Should Allow Entering Vehicle to Clear the Through Traffic Lane.

19.5.2 Designing for Cross Slope Compliance

Sidewalks crossing driveways must comply with the same cross slope requirements (no more than 2.0 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, **Figure 19-19** to **Figure 19-23**. Refer to **TxDOT Roadway Standards - Pedestrian Facilities Curb Ramps** for additional guidance on driveways crossing a sidewalk.

Options 1 and 2: These options are preferred, as they maintain the same sidewalk elevation as the approaching sidewalk and reinforce motorist stopping and yielding behavior. In the case of a setback sidewalk, a width of 4 ft or greater is preferred for the parkway planting to provide suitable apron slopes. See **Figure 19-19** and **Figure 19-20**.

Option 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 design can be designed for the sidewalk setback to be long enough to accommodate a car length so as to not prohibit through movement on the roadway. See **Figure 19-21.** At locations with frequent or closely spaced driveways, this option may result in a meandering sidewalk alignment which is not desirable or increased ROW needs.

Option 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 distance between the flowline of the gutter and the sidewalk. See **Figure 19-22**. Like option 3, this can result in a meandering sidewalk alignment which is not desirable or increased ROW needs. This option is preferable to Option 5.

Option 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. See **Figure 19-23**.

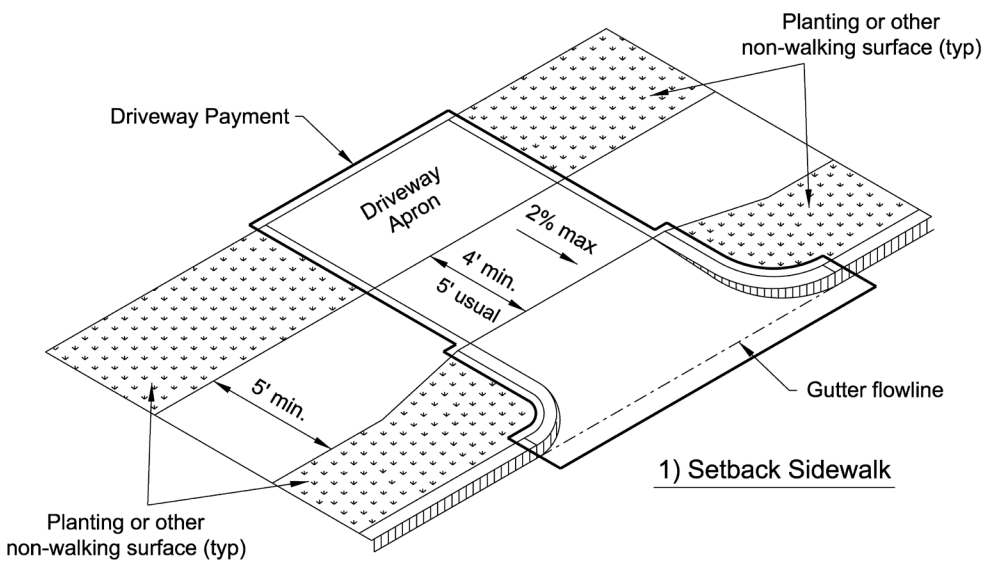


Figure 19-19: Option 1: Setback Sidewalk.

(Preferred design)

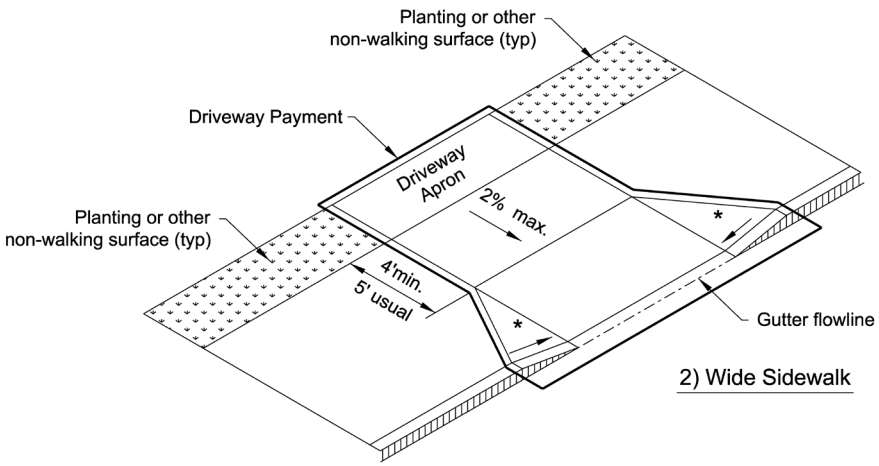


Figure 19-20: Option 2: Wide Sidewalk.

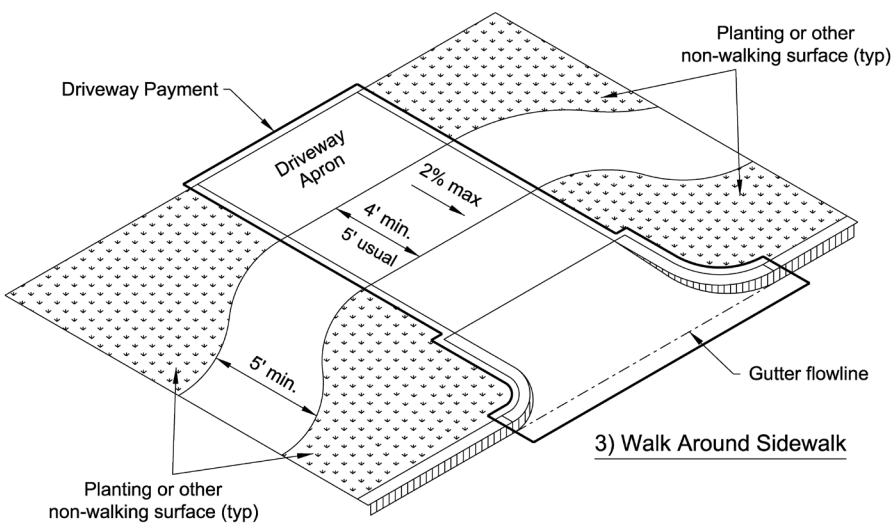


Figure 19-21: Option 3: Walk-Around Design at Driveways.

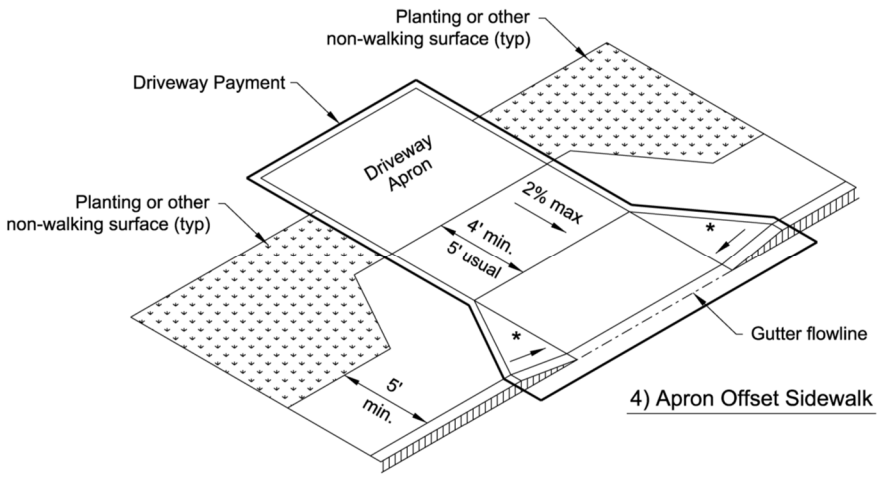


Figure 19-22: Option 4: Apron Offset Sidewalk. (Preferable over Option 5 in Figure 19-23).

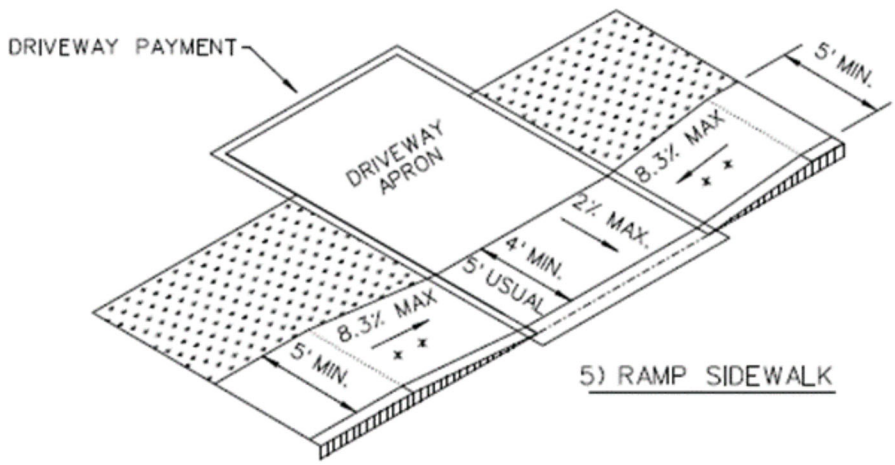


Figure 19-23: Option 5: Ramp Sidewalk.

19.6 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.

19.6.1 Pedestrian Crossings Principles

19.6.1.1 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, thus 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 **Section 13.9.1** for Minimum Intersection Turning Radii.

19.6.1.2 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.

19.6.1.3 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.

19.6.1.4 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; and
- 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.

19.6.1.5 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.

19.6.1.6 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.

19.6.2 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 19-24A** 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. This table provides an initial set of countermeasures options to consider. Not all countermeasures listed within the cell may be necessary to achieve the desired safety goals, however, there are often crossings where the application of multiple countermeasures can be beneficial such as the provision of a marked crosswalks, RRFB, and a crossing island to improve crossing opportunities at multi-lane roadways operating at moderate speeds. On lower speed, lower volume roadways, the application of a marked crosswalk with a complementary pedestrian crossing warning sign may be sufficient where existing night-time lighting levels are adequate.

Figure 19-24B can be used in conjunction with **Figure 19-24A** to evaluate the potential for a countermeasure to address common safety issues.

Other resources which should be consulted to review potential countermeasures not included in these figures 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 webpage; and
FHWA PedBikeSafe webpage.

Roadway Configuration	Posted Speed Limit and AADT								
	Vehicle AADT <9,000			Vehicle AADT 9,000–15,000			Vehicle AADT >15,000		
	≤30 mph	35mph	≥40 mph	≤30 mph	35mph	≥40 mph	≤30 mph	35mph	≥40 mph
2 lanes (1 lane in each direction)	● 1 2 4 5 6	● 1 5 6 7	○ 1 5 6 ● 7	● 1 4 5 6	● 1 5 6 7	○ 1 5 6 ● 7	● 1 4 5 6 7	○ 1 5 6 7	○ 1 5 6
3 lanes with raised median (1 lane in each direction)	● 1 2 3 4 5	● 1 3 5 7	○ 1 3 5 ● 7	○ 1 3 4 5 7	● 1 3 5 7	● 1 3 5 7	○ 1 3 4 5 7	● 1 3 5 7	● 1 3 5
3 lanes w/o raised median (1 lane in each direction with a two-way left-turn lane)	● 1 2 3 4 5 6 7	● 1 3 5 6 7	○ 1 3 5 6	○ 1 3 4 5 6 7	● 1 3 5 6 7	○ 1 3 5 6	○ 1 3 4 5 6 7	● 1 3 5 6	● 1 3 5 6
4+ lanes with raised median (2 or more lanes in each direction)	● 1 3 5 7 8	● 1 3 5 7 8	○ 1 3 5 8	○ 1 3 5 7 8	● 1 3 5 7 8	● 1 3 5 8	○ 1 3 5 ● 7 8	● 1 3 5 8	● 1 3 5 8
4+ lanes w/o raised median (2 or more lanes in each direction)	● 1 3 5 6 7 8	● 1 3 5 6 7 8	○ 1 3 5 6 8	○ 1 3 5 6 7 8	● 1 3 5 6 7 8	● 1 3 5 6 8	○ 1 3 5 6 ● 7 8	● 1 3 5 6 8	● 1 3 5 6 8

Figure 19-24A should be used with Figure 19-24B.

\# 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 Pedestrian hybrid beacon (PHB)**

***It should be noted that the PHB and RRFB are not both installed at the same crossing location.*

This table was developed using information from: Zegeer, C.V., J.R. Stewart, H.H. Huang, P.A. Lagerwey, J. Feaganes, and B.J. Campbell. (2005). Safety effects of marked versus unmarked crosswalks at uncontrolled locations: Final report and recommended guidelines. FHWA, No. FHWA-HRT-04-100, Washington, D.C.; FHWA. Manual on Uniform Traffic Control Devices, 2009 Edition. (revised 2012). Chapter 4F, Pedestrian Hybrid Beacons. FHWA, Washington, D.C.; FHWA. Crash Modification Factors (CMF) Clearinghouse. http://www.cmfclearinghouse.org/; FHWA. Pedestrian Safety Guide and Countermeasure Selection System (PEDSAFE). http://www.pedbikesafe.org/PEDSAFE/; Zegeer, C., R. Srinivasan, B. Lan, D. Carter, S. Smith, C. Sundstrom, N.J. Thirsk, J. Zegeer, C. Lyon, E. Ferguson, and R. Van Houten. (2017). NCHRP Report 841: Development of Crash Modification Factors for Uncontrolled Pedestrian Crossing Treatments. Transportation Research Board, Washington, D.C.; Thomas, Thirsk, and Zegeer. (2016). NCHRP Synthesis 498: Application of Pedestrian Crossing Treatments for Streets and Highways. Transportation Research Board, Washington, D.C.; and personal interviews with selected pedestrian safety practitioners.

Figure 19-24A: Pedestrian Safety Countermeasures Based on Roadway Configuration and Posted Speed Limits and Annual Average Daily Traffic.

Pedestrian Crash Countermeasures for Uncontrolled Crossings	Conflicts at crossing locations	Excessive vehicle speed	Inadequate conspicuity/ visibility	Drivers not yielding to pedestrians in crosswalks	Insufficient separation from traffic
Crosswalk visibility enhancement	✓	✓	✓	✓	✓
High-visibility crosswalk markings	✓		✓	✓	
Parking restriction on crosswalk approach	✓		✓	✓	
Improved nighttime lighting*	✓		✓		
Advance Yield Here To (Stop Here For) Pedestrians sign and yield (stop) line*	✓		✓	✓	✓
In-Street Pedestrian Crossing sign*	✓	✓	✓	✓	
Curb extension*	✓	✓	✓		✓
Raised crosswalk	✓	✓	✓	✓	
Pedestrian refuge island	✓	✓	✓		✓
Pedestrian Hybrid Beacon	✓	✓	✓	✓	
Rectangular Rapid-Flashing Beacon	✓		✓	✓	✓

*These countermeasures make up the STEP countermeasure "crosswalk visibility enhancements." Multiple countermeasures may be implemented at a location as part of crosswalk visibility enhancements.

Figure 19-24B: Countermeasure Effectiveness for Common Safety Issues

19.6.3 Geometric Treatments for Pedestrians at Intersections

19.6.3.1 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 19-25** 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.

19.6.3.1.1 Application

To improve pedestrian safety, refuge islands should be considered where street crossing distances are four travel lanes or greater. See **Figure 19-24A** in **Section 19.6.2** 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 RRFB or pedestrian hybrid beacon (PHB), refuge islands may be used to create two-stage crossings.

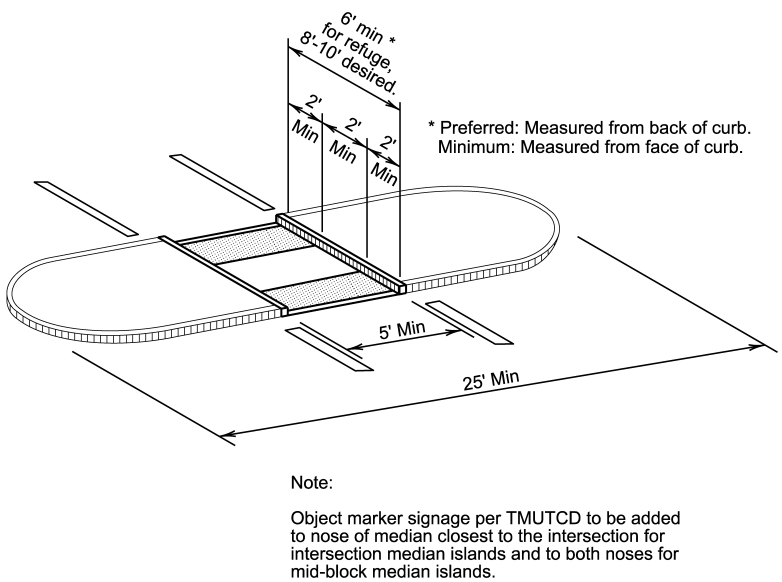


Figure 19-25: A Median Refuge Island.

19.6.3.1.2 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-ft 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 **TMUTCD** and **TxDOT Traffic Standards – Pavement Markings**.

Accessibility: See **Section 19.4.5.4.2.**

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.

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.

19.6.3.1.3 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.

19.6.3.1.4 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.

19.6.3.2 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.

19.6.3.2.1 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.

19.6.3.2.2 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.

19.6.3.2.3 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. **Figure 19-26** shows an example of landscaping within a curb extension.

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 19-26: A Curb Extension with Landscaping.

19.6.3.3 Truck Aprons

A truck apron (**Figure 19-27**) 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.

19.6.3.3.1 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.

19.6.3.3.2 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 with slopes and grades that meet the design requirements for a crosswalk. 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.

19.6.3.3.3 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.

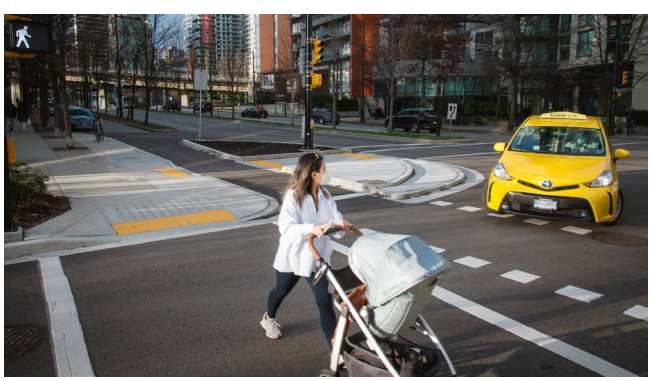


Figure 19-27: A Truck Apron.

19.6.3.4 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.

19.6.3.4.1 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.

19.6.3.4.2 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 may have a stop or yield condition.

19.6.3.4.3 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 for crosswalks and curb ramps.

19.6.3.4.4 Considerations

When designing a raised crosswalk, several other elements should be considered. These include:

- 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; and
- Consult with local emergency services providers regarding placement and design.

19.6.3.4.5 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.

19.6.3.4.6 Application

Raised intersections are applicable in urban areas and towns with higher volumes of pedestrians, at minor intersections, on streets with speeds of 35 mph or less. They may be used at controlled or uncontrolled locations.

19.6.3.4.7 Design Parameters

Accessibility - The junction between the sidewalk and raised crosswalk should meet all applicable accessibility standards for crosswalks and curb ramps.

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.

19.6.3.4.8 Considerations

Raised intersections may have different pavement material to bring attention to the intersection. Ensure that materials within crosswalks meet ADA surface requirements.

19.6.3.5 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. Refer to TxDOT's **Procedures for Establishing Speed Zones Manual** for additional information on posted speed determination.

19.6.3.6 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.

19.6.3.6.1 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.

19.6.3.6.2 Design Parameters

Hardened centerlines should implement the following design parameters:

Materials: Materials for hardened centerlines should be sturdy, stable materials, such as plastic bollards and turning wedges. Refer to **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.

19.6.3.6.3 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.

19.6.4 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.

19.6.4.1 Marked Crosswalks with Uncontrolled Traffic Movements

As discussed in **Section 19.1.3.2**, motorists must stop and yield the ROW 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.

19.6.4.1.1 Application

Marked crosswalks indicate optimal or preferred locations for pedestrians to cross and remind motorists to stop and yield the ROW 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; and
- 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 ROW 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:

- 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
- 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 19.6.2**.

A marked crosswalk is required to legally establish the midblock crossing per **TTC §541.302**.

19.6.4.1.2 Design Parameters

Marked crosswalks must follow **TxDOT Pavement Marking Standards** and further guidance found in **TMUTCD** Section 3B.18. **Figure 19-28** shows an example of a typical marked crossing.

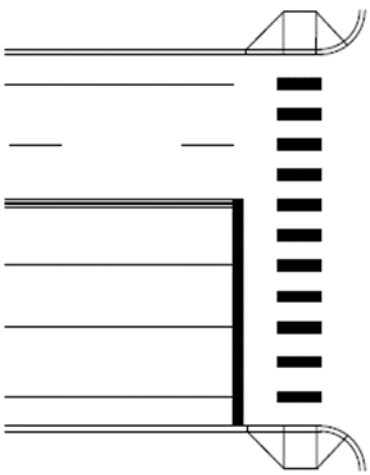


Figure 19-28: A Marked Crosswalk (Controlled Crossing)

19.6.4.1.3 Considerations

Supportive Traffic Control Signs - Regulatory and warning traffic signs (see **Section 19.6.4.3**) 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 crosswalk 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 reduce the need for maintenance. To provide better visibility on concrete roadways, the white crosswalk markings may be outlined with black contrast markings.

19.6.4.2 Stop Bars and Yield Lines

Stop bars (a.k.a. stop lines), 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. Refer to **TMUTCD** Section 3B.16 and Figure 3B-17. **Figure 19-29** provides an example of a marked crosswalk at an uncontrolled intersection.

19.6.4.2.1 Application

See **Figure 19-24A** and **19-24B** for application. Because they are effective at mitigating multiple threat crashes, they should be considered at all marked, unsignalized crossings of multi-lane roadways. They may be used on single lane approaches to marked crosswalks where engineering judgment determines a need.

19.6.4.2.2 Design Parameters

Refer to Section 3B.16 of the **TMUTCD** and the current **TxDOT Traffic Standards – Pavement Markings** for further design guidance.

19.6.4.2.3 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. Refer to 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.

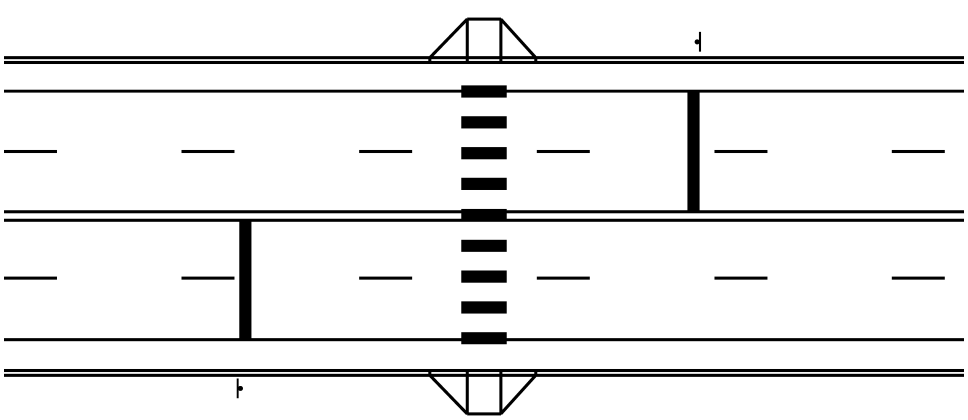


Figure 19-29: Stop Bars at a Marked Crosswalk (Uncontrolled Crossing)

19.6.4.3 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 19.8** below.

19.6.4.4 Pedestrian Crossing Warning Sign Assemblies

19.6.4.4.1 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. See **Figure 19-24A** and **19-24B** for application. The message should reflect the type of crossing (pedestrian (W11-2), school (S1-1), and trail (W11-15 or W11-15a)). See **Figure 19-30**.They should be used at all marked, uncontrolled crosswalks. Refer to **TMUTCD** Section 2C.50 for guidance and standards for use. School zone traffic control is further covered in Chapter 7 of the **TMUTCD**.



Figure 19-30: Pedestrian Crossing Warning Signs (left to right): Pedestrian Crossing, School Crossing, Pedestrian and Bike Trail Crossing, and Trail Crossing.

19.6.4.4.2 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.

19.6.4.5 Pedestrian Crossing Regulatory Sign Assemblies

19.6.4.5.1 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. See **Figure 19-24A** and **19-24B** for application. Refer to Section 2B.11, 2B.12, and Section 3B.16 of the **TMUTCD** for further standards and guidance. **Figure 19-31** shows the current signs at the date this manual was published.

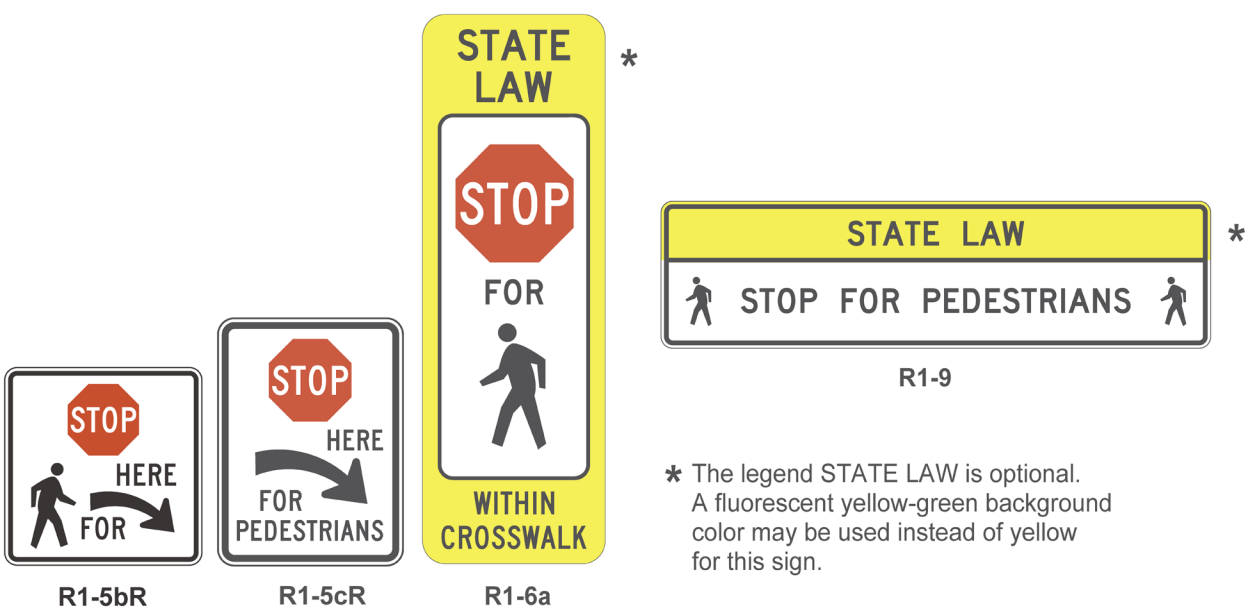


Figure 19-31: Unsignalized Pedestrian Crossing Signs.



Figure 19-32: Pedestrian Stop Sign at an Uncontrolled Intersection.

19.6.4.5.2 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 19-32**).

19.6.4.6 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 19.6.4.3**. RRFBs are generally actuated with a pedestrian push button but may also be activated through passive detection. The pedestrian push button or passive detection device shall activate a speech message that indicates the status of the beacon (e.g., "Flashing lights are on"). If the pedestrian push button is used, it shall not include vibrotactile features as this feature communicates that walk indication is provided for a pedestrian signal.

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 the TRF for specific guidance on the use of RRFBs. As of the publishing of this manual, the link for interim approval of RRFBs can be found in the **RRFB and Pedestrian Hybrid Beacon (PHB) memo (2018)**.

19.6.4.6.1 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. See **Figure 19-24A** and **19-24B** in **Section 19.6.2** for application.

19.6.4.6.2 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. **Figure 19-33** shows an example of an RRFB.

Solar-powered assemblies can eliminate the need for connecting to a power source.



Figure 19-33: Rectangular Rapid Flashing Beacon.

19.6.5 Signalized Crossing Safety Countermeasures

19.6.5.1 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.

19.6.5.2 Standard Signals

19.6.5.2.1 Application

Traffic signals provide benefits to pedestrians by stopping automobile traffic and allowing a designated phase for pedestrians to cross a roadway. See **Section 19.6.2** and TxDOT's **Traffic Signals Manual** for further guidance on the use of traffic signals.

19.6.5.2.2 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.

19.6.5.3 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 TRF Division approval for installation of a PHB for each location.

As of the publishing of this manual, the link for interim approval of RRFBs can be found in the **RRFB** and **PHB memo (2018)**.

19.6.5.3.1 Application

See **Figure 19-24A** and **19-24B** and refer to TxDOT's **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, and 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 and shopping centers); and
- Crosswalk is more than 300 ft. from an existing, traffic controlled pedestrian crossing.

Refer to Chapter 4F of the **TMUTCD** for specific design standards and guidelines.

19.6.5.3.2 Considerations

Designers may justify a PHB in a location that currently has low pedestrian traffic (less than 20 pedestrians per hour) if:

- It can be demonstrated the PHB will attract more pedestrians once installed; or
- Nearby land use changes are expected to generate higher pedestrian volumes.

Figure 19-34 shows an example of a PHB.



Figure 19-34: A Pedestrian Hybrid Beacon.

19.6.5.4 Use of Pedestrian Signal Faces

19.6.5.4.1 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. Refer to Chapter 4E of the **TMUTCD** for guidance and standards, including for accessible pedestrian signals (APS).

19.6.5.4.2 Considerations

Marked crosswalks should be installed at all crossings with pedestrian signals. See **Section 19.6.4.1** for 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. Refer to Section 4E.03 of the **TMUTCD** for further specifications regarding the application of pedestrian signal faces.

19.6.5.5 Actuation

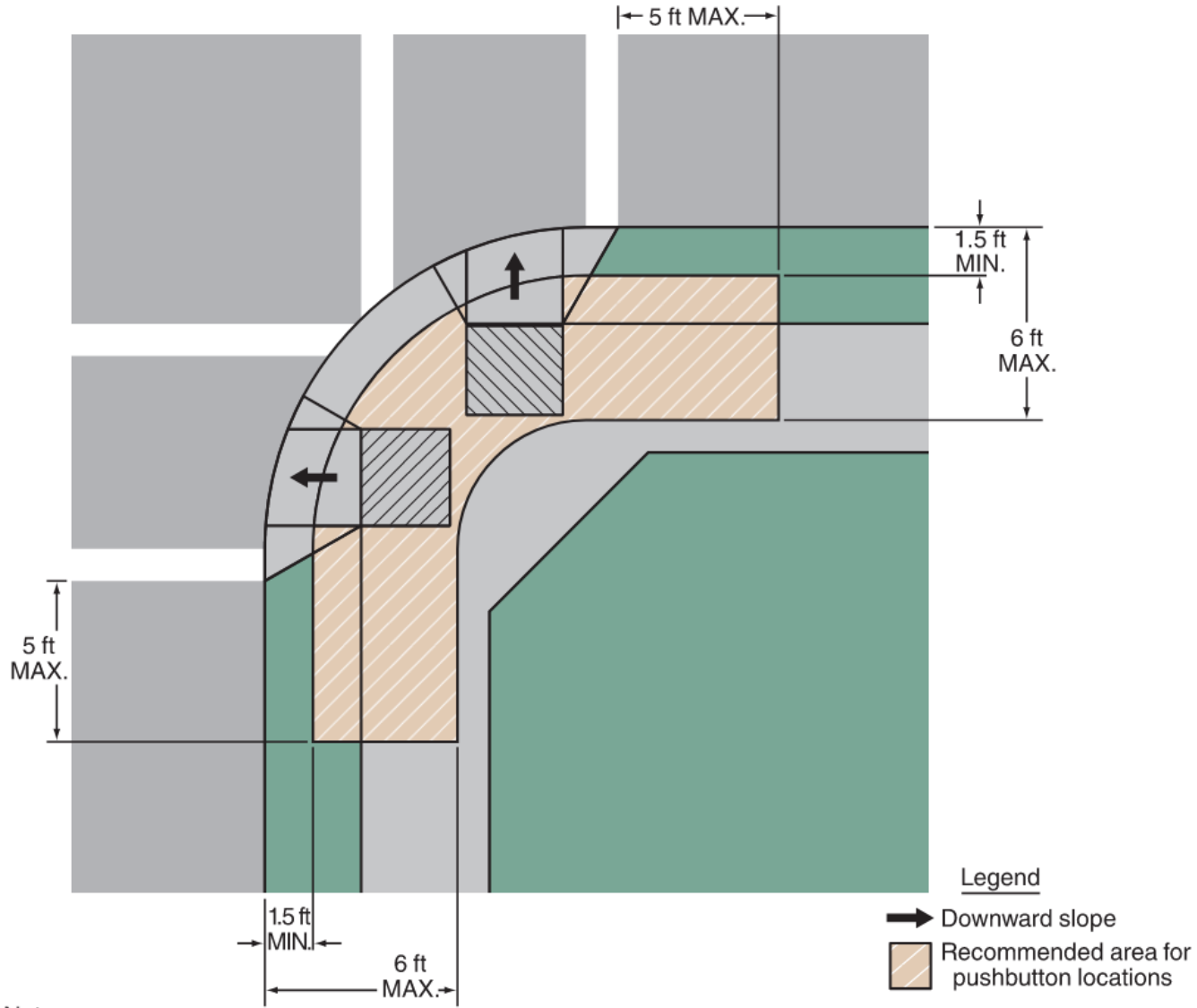
Pedestrian actuation may impact signal or beacon operation in two ways: 1) actuating pedestrian beacons or signal heads that direct pedestrians' crossing behavior, and/or 2) it may also change overall operations of a signal to extend a pedestrian phase only when pedestrians are present, thus allowing them to cross the full width of an intersection leg in one phase.

19.6.5.5.1 Accessible Pedestrian Pushbuttons

APS allow pedestrians to actuate a signal or beacon by pressing a button. APS also provide audible and vibrotactile indications, tactile arrow information communicating the orientation of the crosswalk, and a locator tone to assist pedestrians in finding the pushbutton. APS are required at all signalized intersections with pedestrian signal faces. Some emerging technologies allow pedestrians to activate the signal by waving a hand in the vicinity of the button as well, but this functionality is in addition to the required APS features noted above. Refer to **TMUTCD** Sections 4E.08 through 4E.13 for guidance and standards regarding use of pushbuttons.

19.6.5.5.2 Application

APS push buttons are required at all signals and pedestrian hybrid beacons that include pedestrian signal heads they are also required at RRFBs. See **Figure 19-35** for push button location area.



Notes:

1. Where there are constraints that make it impractical to place the pedestrian pushbutton between 1.5 feet and 6 feet from the edge of the curb, shoulder, or pavement, it should not be further than 10 feet from the edge of curb, shoulder, or pavement.
2. Two pedestrian pushbuttons on a corner should be separated by 10 feet.
3. This figure is not drawn to scale.

Figure 19-35: Preferred Pushbutton Location Area

Source: 2011 TMUTCD

19.6.5.5.3 Considerations

If the button is mounted on its own pole, the pole should not obstruct the pedestrian access route. The pushbutton must always be accessible from the pedestrian access route with a maximum side reach of 10-inches.

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.

At signals and pedestrian hybrid beacons, the use of audible or visual confirmation of detection at signals communicates to pedestrians that the WALKING PERSON indication is forthcoming can improve signal compliance and is required by **PROWAG**. At RRFBs, there is no confirmation message because the flashing lights are activated each time the button is pressed.

Signal supports may be excluded from clear zone requirements. See **Section 4.10.7** and **Table 4-17** for further considerations when placing signal supports within the clear zone.

19.6.5.5.4 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.

19.6.5.5.5 Application

Passive detection may be used to activate pedestrian phases and signal heads and may be used to adjust pedestrian clearance time when a pedestrian is detected. Passive detection does not replace the need for APS pushbuttons as the pushbuttons are still needed to communicate audible and vibrotactile information.

19.6.5.6 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.

19.6.5.6.1 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.

19.6.5.6.2 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**.

19.6.5.6.3 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.

19.6.5.6.4 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. Refer to Section 4E.06 of the **TMUTCD** for further guidance.

19.6.5.6.5 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.

19.6.5.6.6 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.

19.6.5.6.7 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 19.6.5.6** for additional information.

19.6.5.7 Supportive Signs

19.6.5.7.1 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, forces pedestrians to travel longer distances, and increases travel time. If pedestrian crossings at unmarked crosswalks must be restricted, sign types R9-2, R9-3 (**Figure 19-36**), or other appropriate regulatory signs must be installed to face pedestrian approaches of restricted crosswalks. Also note that prohibited crossing locations shall not include curb ramps, and the pedestrian circulation path must be separated from the roadway with landscaping or other non-prepared surface or separated from the roadway by a detectable vertical edge treatment with a bottom edge 15-inches maximum above the pedestrian circulation path.



Figure 19-36: Restricted Pedestrian Crossing Sign.

19.6.5.7.2 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.

19.6.5.7.3 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 VEHICLES STOP FOR PEDESTRIANS (R10-15a) sign should be considered. This sign may be used for both left and right turning traffic.

19.6.6 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 midblock 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.

19.6.6.1 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.).

19.6.6.2 Crossing Treatments

The guidance in **Sections 19.6.3** through **19.6.5** for pedestrian crossing treatments and countermeasures is similarly applicable to

midblock crossings. Refer to the latest **TxDOT Traffic Standards – Pavement Markings** and the **TMUTCD** for guidance on additional signing and markings for midblock crossings.

19.6.7 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. See **Chapter 14** for further guidance on these intersections.

19.6.8 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. Refer to the **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.
    - Refer to **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** requires flangeway gaps have a maximum width of 3-in for tracks that are subject to safety regulations at 49 CFR part 213, issued by the Federal Railroad Administration (FRA), and 2.5-in for tracks not subject to FRA Safety Regulations.
- **Detectable Warning Surfaces** -Detectable warning surfaces (DWS) must be placed on either side of an at-grade crossing only when the crossing is not located in the street. The DWS must be placed a minimum of 6-ft and a maximum of 15-ft from the centerline of the nearest rail.
    - The detectable warning must be, at minimum, 2-ft wide in the direction of pedestrian travel and shall extend the full width of the crossing.
    - Where pedestrian gates are provided, detectable warning surfaces shall be located on the side of the gate opposite the rail. Pedestrian gates shall not overlap detectable warning surfaces.
- **Quiet Zones** - Note that specific standards apply to pedestrian crossing treatments and signage for designated quiet zones. Refer to the Federal Railroad Administration materials related to quiet zones for more information.

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."**

19.7 Overcrossings and Underpasses

19.7.1 Sidewalks for Bridges and Underpasses

19.7.1.1 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 19.10**.

19.7.1.2 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 for stranded motorists or pedestrians who have no alternative route. 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 away from moving traffic. Where shared use paths are used over bridges, the clear width of the shared use path must be wider than the connecting sidewalk to account for the recommended 2-ft shy (1-ft min.) distances from adjacent railings and barriers.

19.7.1.3 Underpasses

Pedestrian facilities such as sidewalks must be provided at vehicular underpasses. Where underpasses are constructed without pedestrian facilities, designs 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 **Section 4.8.6** 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.

19.7.2 Railings and Handrails for Pedestrian Facilities

Railings and handrails can improve safety and comfort for pedestrians on bridges/overpasses and underpasses, and on pedestrian/bike bridges and pedestrian/bike underpasses.

In cases where railings or handrails are provided on both sides of a sidewalk, 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. See **Section 18.4.1** for minimum clear widths on shared use paths.

Additionally, pedestrians' natural behavior is to shy away from vertical elements, such as railings or walls adjacent to a pedestrian route. 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.

19.7.2.1 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 19.12**. Refer to Chapter 3 Section 2 of TxDOT's **Bridge Railing Manual** and Chapter 3 Section 1 of TxDOT's **Bridge Design Manual - LRFD** for further guidance on the use of railings on bridges. **Figure 19-37** shows an example application of railing to treat a steep drop off.



Figure 19-37: A Railing as a Barrier Against a Steep Drop-Off for a Sidewalk Facility at an Overpass.

19.7.2.2 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 height is 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. Refer to **TAS** Section 505 for additional design criteria related to handrails.

19.7.3 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. **Figure 19-38** shows an example of a pedestrian underpass.

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);
- 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); or
- Grades on either side of the proposed bridge or underpass are conducive to building the structure.



Figure 19-38: 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 the **AASHTO LFRD Guide Specifications for Design of Pedestrian Bridges**.

19.7.3.1 Pedestrian/Bike Bridges

19.7.3.1.1 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 2.0 percent slope measured parallel and perpendicular to the path. Assuming a full 30-in rise at 8.3 percent and a 5-ft landing at 2.0 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. See **Section 18.4.1** for recommended and minimum widths on shared use paths. 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 **Section 4.8.6** for vertical clearance requirements for pedestrian crossover structures.

Lighting - When provided, lighting for pedestrian and bike bridges should allow users to see one another in nighttime conditions to avoid collisions and improve the perception of personal security.

19.7.3.2 Pedestrian/Bike Underpasses

19.7.3.2.1 Design Parameters

Grade - Underpasses must comply with ADA guidelines(not exceeding a 5 percent running slope or 2.0 percent cross slope.) Where space

constraints result in necessary steeper grades, underpasses must still comply with ADA requirements for ramps with landings. Underpasses grades and lengths should allow for clear sightlines at the approach and through the underpass.

Width - Width recommendations are 14- to 16-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 - When provided, lighting for pedestrian and bike underpasses should allow users to see one another in nighttime conditions to avoid collisions and improve the perception of personal security.

19.8 Work Zone and Temporary Traffic Control Pedestrian Accommodations

43 TAC §22.12 states that traffic control plans shall address pedestrian traffic during individual phases of construction and operation. The **TMUTCD** Part 6 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)** 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 19-39** depicts an example layout for a sidewalk detour.

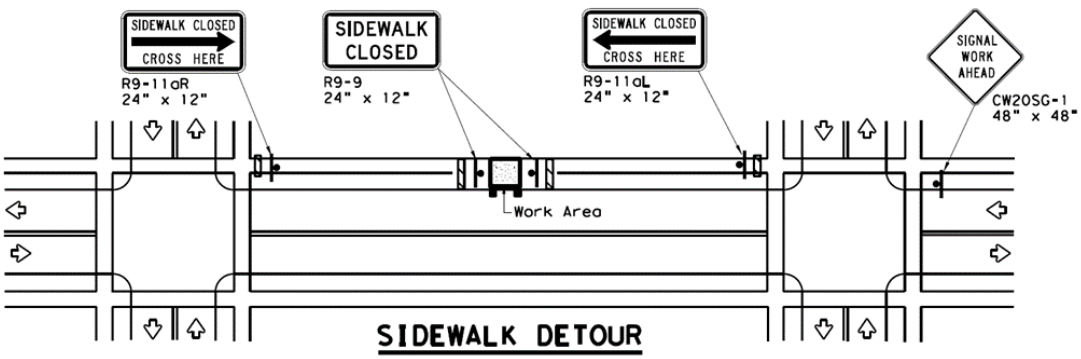


Figure 19-39: Sidewalk Detour Details.

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 must 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.)

Where pedestrian detours are implemented, appropriate signing must be provided in advance of decision points to allow pedestrians to make appropriate decisions about navigating the space. **PROWAG** also requires that motion activated signs that convey the same information as the printed sign be provided to ensure that pedestrians with vision disabilities are provided the guidance needed to navigate the temporary pedestrian route.

Where transit stops are disturbed as part of construction or maintenance activity, an alternate accessible transit stop must be provided.

Refer to **TMUTCD** Section 2D.50 for additional guidance on the placement of pedestrian oriented wayfinding signage.

19.9 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 at commercial districts, near parks and schools, and along transit routes, are most in need of lighting.

Lighting should be considered for all location where pedestrian crossing is expected such as signalized intersections, uncontrolled intersections, and mid-block locations,. At controlled and uncontrolled intersections, luminaires should be located to provide appropriate light levels as recommended by **ANSI/IES RP-8-22** or the **FHWA Lighting Handbook**.

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 TxDOT's **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 to light both the roadway and the sidewalk. All street lighting should not include 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 19.2.2.1.**

Design of lighting systems should be coordinated with local utilities.

19.10 On-Street Parking

Where on-street parking is provided, it must 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. All on-street parking spaces shall be identified with a sign displaying the **International Symbol of Accessibility**.

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.

19.10.1 Parallel Accessible Parking

Where space between the curbline and ROW line is greater than 9-ft wide, parallel accessible spaces must be provided which are 24-ft long by 13-ft wide. See **Figure 19-40**. These dimensions will accommodate both the space for the vehicle as well as the accessible aisle; however, an accessible aisle shall not be marked to ensure that drivers can position the parked vehicle in a manner that best accommodates their loading and unloading needs. This parking space must be connected to the sidewalk by a curb ramp which shall not reduce the dimensions of the parking space and shall be located at either end of the parking space. Where parallel accessible spaces adjacent to sidewalks less than or equal to 9-ft are provided, the parking space width may match the adjacent parking space widths but the accessible space must be located nearest a crosswalk at the end of the block or nearest a midblock crosswalk so that access to the street can be served by these adjacent curb ramps. For all accessible parking spaces, the center 50% of the parking space along the sidewalk must be free of obstructions (e.g., parking meters and sign posts) to allow for wheelchair ramp deployment.

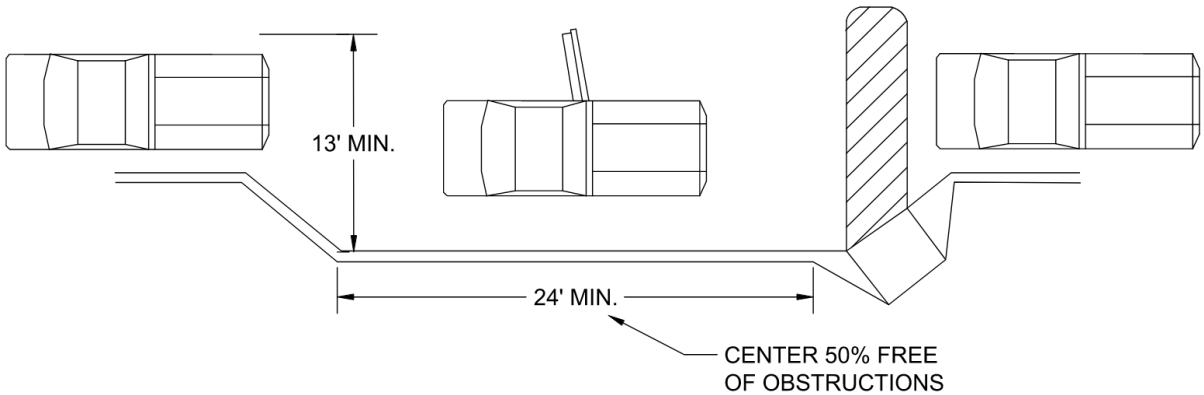


Figure 19-40: Accessible Parallel Parking

19.10.2 Perpendicular Accessible Parking

Perpendicular accessible spaces require a van accessible aisle with a minimum width of 8-ft extending the full length of the parking space. The access aisle must be marked, must be at street level, and shall not encroach on the traveled way. Two adjacent perpendicular or angled accessible spaces may share a common access aisle where front and rear entry parking is permitted. Where an access aisle serves only one parking space and parking is restricted to either front entry or rear entry orientation, the access aisle shall be located on the passenger side of the vehicle. The **TxDOT Traffic Standards – Pavement Markings** provides specific guidance for pavement markings and signing for accessible parking.

19.10.3 Angled Accessible Parking

Angled accessible spaces must be a minimum of 11-ft wide with a minimum 5’ wide accessible aisle extending the full length of the parking space. The access aisle must be marked, must be at street level, and shall not encroach on the traveled way.

19.10.4 Curb Ramp

Curb ramps that access accessible parking spaces are not required to include detectable warnings unless they also serve pedestrian street crossings. The designer has the option of providing detectable warnings or not. **Figure 19-41** provides an examples of the curb ramp options.

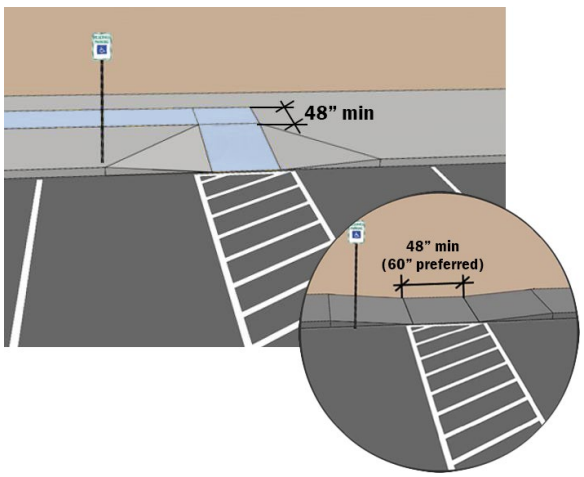


Figure 19-41: Perpendicular Parking Access Aisles, Showing A Curb Ramp With Flares (Top) And A Parallel Curb Ramp (Bottom).

19.11 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 (see **Figure 19-42** to **Figure 19-45**):

- Slopes between 1V:3H and 1V:2H, with a drop of 6-ft or greater (**Figure 19-42**);
- 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 19-43**);
- Slopes of between 1V:2H and 1V:1H, with a drop of 4-ft or greater (**Figure 19-44**); or
- Slopes of 1V:1H or steeper, with a drop of 1-ft or greater (**Figure 19-45**).

Railings used for drop-off protection should be 42-in minimum height. Railing construction should be consistent with the **AASHTO LRFD Bridge Design Specifications** per TxDOT's **Bridge Railing Manual**.

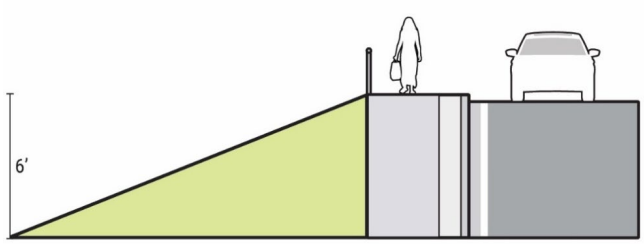


Figure 19-42: Slopes Between 1V: 3H and 1V:2H With a Drop of 6-ft or Greater.

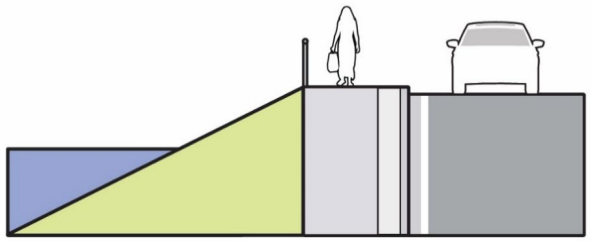


Figure 19-43: Slopes of Between 1V:3H and 1V:2H, Adjacent to a Parallel Body of Water or Other Substantial Obstacle.

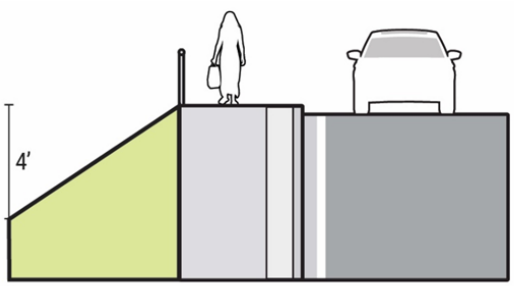


Figure 19-44: Slopes of Between 1V:2H and 1V:1H, with a Drop of 4-ft or Greater.

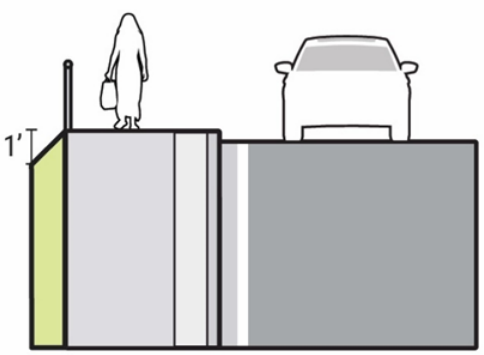


Figure 19-45: Slopes of 1V:1H or Steeper, with a Drop of 1-ft or Greater.

19.12 Additional Considerations

19.12.1 Personal Security

People may choose not 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 ROW 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 sight lines between the sidewalk and roadway through appropriate choice of the height and type of plantings.

19.12.2 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. Refer to TxDOT's **Landscape and Aesthetics Design Manual** for further guidance.

19.12.3 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.**

Chapter 20
Motorcyclist Design Considerations

20.1 Overview

This chapter discusses motorcycle-specific road safety advice to aid roadway designers when preparing plans for construction. The chapter is specific to motorcycling safety, the goal being to ensure that issues faced by motorcyclists are known when roadway plans are designed. The chapter is divided into three sections providing information on the following highway features:

- **Pavement Features** - Pavement features can include utility covers, pavement markings, and surface treatments, all of which can affect motorcycle stability.
- **Roadside Features** - These are physical objects in the cross section that can increase the severity of a collision such as trees, vegetation, posts, and longitudinal barriers.
- **Highway Features** - These are specific elements of the highway geometry such as horizontal curves and intersections where motorcyclists may be vulnerable to a collision.

This chapter describe various issues that impact motorcycle safety. Relevant mitigation measures are then identified that can help reduce the risk to motorcyclists. The chapter includes the following sections and subsections:

20.1 Overview

20.2 Pavement Features

- 20.2.1 Metal Covers;
- 20.2.2 High Friction Surfacing;
- 20.2.3 Removal of Pavement Markings; and
- 20.2.4 Pavement Markings.

20.3 Roadside Features

- 20.3.1 Vegetation;
- 20.3.2 Signs and Other Road Furniture;
- 20.3.3 Longitudinal Barriers;
- 20.3.4 Shoulders; and
- 20.3.5 Clear Zones.

20.4 Highway Features

- 20.4.1 Horizontal Curves (General);
- 20.4.2 Horizontal Curves (Specific);
- 20.4.3 Intersections;
- 20.4.4 Roundabouts; and
- 20.4.5 Signs Related to Motorcyclists.

20.5 Motorcycle Safety During Construction

- 20.5.1 Pavement; and
- 20.5.2 Construction Maintenance.

20.6 Summary of Recommendations

20.2 Pavement Features

Pavement features include aspects such as utility covers, pavement markings, and different surface treatments. Each can present a particular challenge for motorcyclists as the materials can often have a different and lower skid resistance to the surrounding roadway. A difference in surface materials and skid resistance can result in a motorcyclist losing control particularly if the feature is located in an area that requires motorcyclists to brake or turn. This issue can be further compounded in wet conditions.

The following features and potential design considerations are considered in more detail within this section:

- Metal covers;
- High friction surfacing;
- Removal of pavement markings; and
- Pavement markings.

20.2.1 Metal Covers

Motorcyclists are vulnerable to skidding on metal covers which typically have a lower skid resistance than the surrounding pavement. Where there is a sudden change in road surface properties, it can lead to stability issues for motorcyclists.

The locations that are most problematic are where a cover is located within the riding line in areas where a motorcyclist is likely to brake, turn, accelerate and/or change lanes. This can be a particular issue on curves, intersections, and roundabouts.

Treatment: Historically, metal covers for utilities and storm drainage have been located within the pavement. However, designers should strive to ensure that metal covers are not designed for new pavements, including main lanes, shoulders, turn lanes, and median openings of high speed and/or arterial roadways. If these facilities already exist in the pavement, they should be relocated where practicable.

Where there are existing metal covers that cannot be practically moved, then they should be treated with a surface or coating so that the polished skid resistance value (PSRV) matches the surrounding pavement.

Details for metal coverings should also be reviewed to ensure that the tops are flush with the adjacent pavement and do not present a height differential.

20.2.2 High Friction Surfacing

High friction surface treatment (HFST) provides an enhanced level of skid-resistance and is typically applied on high speed roadways, the approaches to signalized intersections, roundabouts, small radius curves, and pedestrian crossings.

Where there is a transition from a HFST to a standard road surface, motorcyclists will experience a reduction in grip between the tires and the pavement. This may not be a problem on straight sections of pavement but, if the HFST ends on a bend, a change in the road surface friction can destabilize a motorcyclist.

Where HFST is terminated is based partly on engineering judgement, therefore it is important that the surfacing extends to a point where the hazard has diminished. An example of this is on the exit from a horizontal curve where the extension of the HFST by a matter of feet can move the transition point beyond the bend and onto a straight section of pavement.

This can be a simple but effective measure to assist motorcyclists.

Treatment: Designers need to specify appropriate friction characteristics for pavement surfacing depending on the nature of the road environment at each location. This includes where motorcyclists and other road users are accelerating, braking, and cornering.

HFST may be recommended in the following scenarios:

- Approaches to and across intersections, approaches to roundabouts and traffic signals;
- Approaches to pedestrian crossings and other high-risk situations;
- Gradients greater than 5% and longer than 150-ft; and
- Horizontal curves on high speed roadways with the minimum radius at normal crown and near or at the minimum radius for full superelevation, where the side forces are at their highest.

20.2.3 Removal of Pavement Markings

As highway layouts change, pavement markings are repeatedly renewed, or surface dressing is applied over old markings. This can lead to an accumulation of pavement markings and surface treatments which can destabilize motorcyclists particularly if this results in standing water. Similarly, masking of redundant markings can also result in areas of surfacing with different skidding resistance.

Treatment: When placing new pavement markings, it is best practice to remove the existing pavement markings entirely to eliminate the risk of a build-up of material. When grinding is used to remove existing markings, the grinding should be specified in such a way that the ground pavement surface does not appear to still be pavement markings in wet or dark conditions.

20.2.4 Pavement Markings

Pavement markings may have lower skid resistance than the pavement surface, which may impact motorcyclists or bicyclists more than vehicles with 3 or more wheels. Larger marking area, such as crosswalks and stop bars, may pose even greater issues.

Treatment: When introducing pavement markings, it is good practice to locate them away from where motorcyclists are likely to be accelerating, braking or turning. Pavement markings with an improved skid resistance similar to that of the adjacent roadway could be used where the road marking is considered necessary.

If thermoplastic markings are used, designers should check the specifications to ensure that a sufficient quantity of glass beads or some other rough surface additive are included to

provide some skid resistance in wet weather as well as maximizing reflectivity.

A designer may also investigate manufactured roadway striping that is adhered to the pavement surface and can be specified with high-friction surfaces that are resistant to wear.

There are some disadvantages around application of Methyl Methacrylate (MMA), including 'greying' of the marking over time due to traffic and the retroreflective performance. Designers need to take a balanced approach when considering pavement markings and use different types of road marking dependent on the site and the potential skidding risk to motorcyclists.

20.3 Roadside Features

Roadside features are objects in the cross section such as trees, vegetation, posts, culvert headwalls, and longitudinal barriers which can present a risk to motorcyclists. This is often because motorcyclists become separated from their vehicle in the event of a collision or loss of control and are then at risk of coming into conflict with a feature in the roadside.

Collision data between 2015 and 2019 , shows that 60% of all collisions which involved a motorcyclist colliding with a feature in the roadside, such as signs, lighting columns, telephone or electricity poles, and trees, resulted in fatal or serious injuries. For the same period, collisions involving cars colliding with a roadside feature resulted in 30% fatal or serious injuries. This demonstrates the vulnerability of motorcyclists when roadside features are present.

Designers need to understand and apply good road safety practice to manage roadside risks for all road users. **Chapter 17 Roadside Safety Design and Roadside Safety Hardware** can be consulted for guidance. Additional considerations for motorcycles beyond the guidance of that chapter may also be considered.

The designer may also refer to the recent TxDOT Research Project 0-6994, Develop a Retrofit Design for Guard Fence System to Enhance Motorcycle Safety, as well as the resultant Motorcycle rubrail standard available on the Roadway Standards webpage.

20.3.1 Vegetation

While vegetation is often planted when trees and hedgerows are saplings, eventually they will mature and become an immovable object. As such, vegetation in the cross section can create a risk for motorcyclists in the event that they lose control and leave the roadway.

Mature vegetation can also have an impact on sight lines and restrict other road users view of approaching motorcyclists and/or limit road users' awareness of the approaching road layout.

Treatment: Road landscapes should aim to fit with the local landscape character, and this will include reflecting the predominant vegetation patterns and species. This is important not only for reducing the impact of the road corridor on the landscape but also in benefiting the road user experience.

However, planting design also needs to consider safety requirements as well as longer term maintenance, management of these areas, the nature of the road corridor that it is intended for, and operational requirements.

Designers can reference the **Landscape Manual** or see **Chapter 17** for specific guidance.

Designers should ensure that the immediate area adjacent of the traveled way is free from vegetation which could result in serious or fatal injuries to motorcyclists.

20.3.2 Signs and Other Road Furniture

Cross sections can have signs, lighting columns and other road furniture within them, which if poorly located, can result in injuries if struck by a motorcyclist. Introducing these structures with breakaway supports can provide an important tool in reducing the severity of vehicle related collisions, however the standard for testing the safety of features does not take into account motorcycles or motorcyclists.

As a result, designers need to consider that breakaway supports may not perform in the same manner when hit by a motorcycle or motorcyclist. These supports may not shear or deflect as intended, potentially resulting in higher severity injuries for this group of road users. The use or need for this infrastructure should be carefully considered.

Treatment: When developing highway schemes that incorporate roadside furniture the positioning of these features needs to be carefully considered.

Designers need to understand and apply good road safety practice to manage roadside risks for all road users.

The principle of remove, relocate, reduce, and protect the roadside feature ensures that the concerns of motorcyclists are included when introducing road equipment support structures into a design.

20.3.3 Longitudinal Barriers

The objective of installing a longitudinal barrier is to contain and redirect errant vehicles so that they do not cross medians into the path of other vehicles, or impact roadside features. Most barrier systems have been designed and tested to account for passenger vehicles and not motorcyclists. A recent study showed that motorcyclists accounted for 40% of all fatalities resulting from a guard fence collision compared to 60% for occupants of all other vehicle types. (Gabier et al., 2022) Motorcyclists are clearly overrepresented in impacts given they comprise only 3% of the registered vehicles.

Motorcyclists are vulnerable to injury when they strike longitudinal barriers. Motorcyclists separated from their motorcycles can slide beneath a guard fence beam because the gap beneath the beam can allow a motorcyclist to impact the unprotected posts. When a seated motorcyclist impacts a guard fence and is ejected, they can slide across the guard fence and impact the top of the posts and beam edge. Concrete barriers on the outside edge of elevated structures present fall risk to the ejected motorcyclists.

Treatment: Designers need to apply good road safety practice to manage roadside risks for all road users. The first consideration should be if the roadside feature that requires longitudinal barriers can be removed or avoided entirely.

Where a barrier is identified as required for a new design, and where there are specific risks to motorcyclists, it is important that the type of barrier chosen minimizes the risk to these road users. Specific barrier treatments have been developed and tested (Schulz et al., 2023 and Dobrovolny et al., 2018) to help mitigate risks to motorcyclists. A properly designed rubrail

element beneath a guard fence beam can reduce the potential for interaction of a sliding motorcyclist with a post. A cap rail mounted on top of the guard fence can mitigate risk of an ejected motorcyclists interacting with the top of the posts or beam edge. Refer to **the Roadway Design Standards webpage** for a crash tested rubrail and caprail system. A containment fence mounted behind a concrete barrier can help capture an ejected motorcyclist to prevent a fall off an elevated structure.

Chapter 17 has additional guidance with respect to Roadside Safety hardware applications that can mitigate risks to motorcyclists.

The provision of an add on feature to a vehicle barrier is a risk-based approach usually evidenced through collision data or where other mitigation measures cannot be introduced. A recent evaluation of Texas motorcycle fixed object crashes shows a high risk when motorcyclists impact bridges and guard fences on curved roadways (Schulz et al., 2023). High risk is also indicated for motorcyclists impacting concrete barriers in high-speed locations (≥50 mi/h) and AADT ≥928.

Such 'add on' products need to be compatible with the restraint system to which it is being attached.

20.3.4 Shoulders

The presence and condition of shoulders may affect motorcyclist safety. A lack of shoulders offers a motorcyclist no place to take an evasive riding action if necessary. A stalled motorcyclist on a roadway without a shoulder or with only a narrow shoulder is especially vulnerable to moving traffic on the adjacent lane. A wide shoulder provides an errant motorcyclist recovery space before impacting a fixed roadside object. Paving currently unpaved shoulders so they can be used for these purposes is a recommended method of assisting in motorcycle recovery and avoidance maneuvers. Plans should ensure that paved shoulder edges are tapered to the unpaved area and do not present a dropoff.

Shoulder width and AADT are critical factors that influence motorcycle crash frequency. In rural environments, locations with narrow shoulders provide little recovery space for vehicles negotiating a horizontal curve. Rural roadways with shoulder widths of 6-ft or less will have approximately 52 percent greater likelihood of crashes occurring at horizontal curve locations.

20.3.5 Clear Zones

Clear zone widths in the **AASHTO Roadside Design Guide**, which is the basis for guidance in the RDM, are calculated using a passenger car or truck as the design vehicle. A clear zone for a motorcycle might need to consider a motorcyclist who has been thrown from his or her bike, making it uncertain if the AASHTO clear zone widths are adequate. A wider clear zone is better because it allows a motorcyclist more distance to decelerate before striking a roadside feature.

Motorcycles differ from other vehicles in that they overhang their wheel track by about 1-ft to 2-ft on each side and because they lean to change direction, which markedly alters the

clearance they require when travelling upright. Failure to account for these characteristics may result in riders colliding with poles, signs, or fences that are placed too close to the edge of the pavement.

When objects cannot be located outside the clear zone, agencies have recommended a variety of methods to minimize risk to motorcyclists including:

- Using flexible or breakaway signs and poles;
- Mounting street signs on light poles rather than on a separate signpost;
- Locating objects behind protective barrier; and
- Using motorcycle-friendly crash barriers.

20.4 Highway Features

20.4.1 Horizontal Curves (General)

On horizontal curves, motorcyclists will generally adopt a line that follows the vanishing point. However, if a roadside object such as a tree or field gate interrupts the vanishing point then motorcyclists may veer towards that object. This can then result in a loss of control collision and as already highlighted within the chapter, when a motorcyclist is involved in a collision with a roadside feature it can result in fatal or serious injuries. To reduce this risk, motorcycle trainees are often taught that 'Where You Look Is Where You Go' (WYLIWYG).

Treatment: Where there are safety concerns of road users losing control on a curve, the WYLIWYG principle can be introduced. WYLIWYG uses a series of flexible delineator post to draw the focus of motorcyclists and drivers to the vanishing point of the curve and prevents distraction by roadside objects.

Flexible delineator posts are placed in the margin at regular intervals, extending both around the curve as normal and also for some distance after the curve until the vanishing point starts to move away from the rider's view into the subsequent tangent.

WYLIWYG is most commonly introduced on unlit rural roads.

Designers need to give consideration to passive safety when proposing the installation of flexible delineator post. Providing flexible delineator post can have maintenance implications, since it is essential that the posts remain visible to maintain their effectiveness.

20.4.2 Horizontal Curves (Specific)

Transportation agencies in the United States typically use circular curves at roadway horizontal curve transition locations. A motorcyclist adapts to the roadway curvature by reducing speed upon entry to the curve. This speed change is often accompanied by a path correction where the motorcycle may enter the curve on the outside of a travel lane, shift to the inside of the lane as the vehicle traverses the curve, and then return to the outside of the lane as the vehicle exits the curve. This path correction often occurs at the same time as the motorcycle accelerates for the length of the curve. Drivers of four-wheel vehicles make similar adjustments to their travel path.

Horizontal curves where the radii values are less than 500-ft are 40 times more likely to have motorcycle-to-barrier crashes than when the radii are 2,800-ft or greater.

Horizontal curves with a radius of 820-ft or less can expected to increase crash frequency by a factor of 10.

Locations with identical radii, roadways with longer curves, larger AADT values, and isolated curve configurations (i.e., curves not located within 300-ft of another curve end) are stronger candidates for the placement of motorcycle-to-barrier crash countermeasures.

When compared with flat curves (radius greater than or equal to 4,000-ft), sharp curves with a radius less than 1,500-ft could be expected to increase the probability of a fatal or serious injury crash by approximately 8 percent. Moderate curves did not have any significant trend. Locations with reverse curves were affiliated with an increase of approximately 6 percent in fatal or serious injury crashes.

Risk for single-motorcycle crashes decreased as the horizontal curve radius increased. This equated to a motorcyclist having almost five times the risk of a crash on horizontal curves with a radius of 1,500-ft or less compared to risk on a similar road with a straight section. This amount reduced to about two times the risk for moderate curve radii (greater than 1,500-ft but less than or equal to 3,000-ft). For flatter curves with radii greater than 3,000-ft, the risk was 1.88 times that observed for a straight roadway segment. Researchers define a straight segment as one with a radius greater than 20,000-ft.

The placement of reverse curves resulted in more crashes than at locations without reverse curvature. This observation reversed the findings from the 2017 study by the same core group of authors. The authors hypothesized that this reversal was due to inclusion of an interaction term in the statistical model that accounted for curve radius as well as curve type.

Higher speed limit and presence of auxiliary lanes and intersections were all associated with higher crash frequencies. The researchers also determined that paved roads and shoulders were associated with higher speeds and therefore increased the likelihood of crashes. In contrast, they observed that narrower roads tended to be associated with fewer crashes. The researchers hypothesized that this observation may be because narrower roads tend to be lower-performing local roads and more riders on these facilities may be local riders who are familiar with the narrower road.

Crash Modification Factors for Motorcycles:

General equation except for rural two-lane:

$$CMF_R = \left(\frac{R}{5000}\right)^{-0.208}$$

Empirical CMFs specific to rural two-lane:

3.27	$R \leq 1000$ ft
2.98	1000 ft $< R \leq 2000$ ft
1.82	2000 ft $< R \leq 10{,}000$ ft

20.4.3 Intersections

The main challenge for motorcyclists at intersections is motorists failing to register an approaching motorcyclist. This is often because of the small frontal aspect that a motorcyclist presents head on and other road users' poor judgement of motorcycle approach speeds.

Treatment: To reduce the risk to motorcyclists, clear sight distance needs to be maintained at intersections so that driver sight lines are optimized, and motorcyclists do not become 'masked' by roadside features. This includes restricting parking on the approaches to intersections and positioning road furniture/vegetation out of the sight triangle.

Inter-visibility between drivers and motorcyclists can be improved by the minor approach of an intersection being perpendicular to the major road. This also helps to reduce the likelihood of the view of the driver being obscured by the vehicle's door pillar and consequently increasing the risk that the driver fails to see an approaching motorcycle.

20.4.4 Roundabouts

Roundabouts can present a number of risks for motorcyclists since accelerating, braking and maneuvering are all necessary. As highlighted earlier within the chapter, this can be compounded when there are large amounts of pavement markings and metal plates in the pavement which can introduce variations in skidding resistance.

Collisions at roundabouts typically involve:

- Drivers failing to yield;
- Drivers looking but failing to see a motorcyclist;
- Misjudgment of motorcyclist's speed; and
- Tight curves on entry.

Treatment: Motorcycle collisions at roundabouts can be partly mitigated by clear signing and markings, by limiting visibility to the left until 50-ft before the entry using suitable screening (**NCHRP Report 1043, Exhibit 9.17**), and by ensuring that the lane assignments of a multilane roundabout are self-explanatory.

Designers should avoid sharp curves on roundabout approaches and entries as these can cause loss of control collisions for motorcyclists. Reverse curves intended to calm traffic should be designed to keep side forces minimal. Tangent sections between consecutive horizontal curves of 75-ft or greater length are recommended to allow a motorcyclist to regain balance before navigating the second horizontal curve. Designers should also ensure that surfacing materials and pavement markings have suitable skidding resistance, especially at the crosswalks and yield entry lines where YIELD words, yield symbols, and dashed lines will be present to delineate the yield point at a roundabout entry.

20.4.5 Signs Related to Motorcyclists

The **TMUTCD** is the standard for traffic control devices on roads open to public travel. See **TMUTCD** Section 2C.33 for warning signs and plaques to be used for motorcyclist. Conditions that these signs came be used to mitigate include:

- Loose gravel;
- Rough road;
- Grooved pavement;
- Textured or brick pavement; or
- Metal bridge decks.

Some signs and plaques have special word message signs for situations where roadway conditions make it necessary to provide road users with additional warning information and where a standard message is not provided in the **TMUTCD**. As an example, these are the STEEL GRID DECK (W8-V1), EXPANSION JOINTS

(W8-V2), and OPEN JOINTS ON BRIDGE (W8-V3) warning signs.

The need for smooth riding surfaces for motorcyclists should be provided, or advance warning notification given whenever roadways surfaces are disturbed. When plans call for milled or grooved pavement, the specifications should be reviewed so that the resulting surface is fine enough to prevent motorcycles tires from being captured or redirected at high speed.

The special needs and control of motorcyclists should also be considered through a temporary traffic control zone.

Pavement scarring of the roadway exceeding 1/8-inch should be repaired to prevent deterioration of the pavement surface and to provide a smooth surface for motorcyclists.

Metal coverings provide a challenge to motorcyclists when they traverse them unexpectedly in the roadway.

The design of the crossover should accommodate all vehicular traffic including motorcycles, trucks, and buses.

20.5 Motorcycle Safety During Construction

20.5.1 Pavement

During construction, contractors may not maintain existing or temporary pavement surfaces to the same extent that permanent pavement surfaces are maintained. Existing pavement defects such as rutting, potholes, cracks, and “tar snakes”, which are a temporary repair for cracks, can cause the motorcycle operator to lose control. During construction, a contractor may leave milled pavement or vertical asphalt edges that can present risks. Milling machines with smaller teeth that provide for a smoother surface can be specified. Details can require asphalt edges of 1V:3H or flatter.

20.5.2 Construction Maintenance

Debris and dust that might not pose a risk to cars and trucks can be risky to motorcycles. Loose rocks can strike motorcycle operators, dust can obscure vision, and mud from construction vehicles leaving the construction zone and entering pavement can cause friction loss between the motorcycle and the pavement.

20.6 Summary of Recommendations

- Increase the radius of a horizontal curve to provide enhanced visibility.
- Increase the length of a horizontal curve.
- Consider including auxiliary lanes (particularly exclusive right-turn lanes) at locations where motorcycle volume is higher.
- Minimize the roadside features (trees, barriers, and other fixed objects) and widen clear zones with particular attention to the outside of the curve.
- Widen shoulder widths in a manner consistent with the road purpose (e.g., wider for freeways, narrower for two-lane highways). Widen the paved shoulder at curves to allow improved recovery space to avoid roadway departures or MBGF impacts.
- Employ human factors research and experimentation with novel motorcycle warning signs with the potential to reduce crashes, in accordance with the provisions of the **TMUTCD**.
- Maintain smooth driving surfaces.
- Minimize the use of raised pavement markings at locations with frequent lane changes (additional research is needed for snow plowable pavement markings). Evaluate the use of advance stop lines for motorcycles.

Chapter 21
Texas Highway Freight Network, Texas Trunk System, National Highway System, STRAHNET and Hurricane Evacuation Routes

21.1 Overview

This chapter discusses the different networks of highways within Texas that serve specific purposes and may require additional design considerations or higher standards. The following sections provide a general overview of each network as well as a historical summary. All of the networks discussed below can be seen on the **Statewide Planning Map**. The chapter includes the following sections and subsections:

21.1 Overview

21.2 Texas Highway Freight Network (THFN)

- 21.2.1 Vertical Clearance at Structures;
- 21.2.2 Signs, Overhead Sign Bridges (OSB's), Signals;
- 21.2.3 Other Overhead Utilities; and
- 21.2.4 Bicycle and Pedestrian Accommodations.

21.3 Texas Trunk System (TRUNK)

21.4 National Highway System (NHS)

- 21.4.1 History of the National Highway System

21.5 Strategic Highway Network (STRANET)

21.6 Hurricane Evacuation Routes

- 21.6.1 Hurricane Evacuation Lanes; and
- 21.6.2 Contraflow Lanes.

21.2 Texas Highway Freight Network (THFN)

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 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:

- Required on all projects let on September 1, 2020, or later;
- Designated as being on the THFN as shown on the THFN map maintained by TPP; and
- New bridge construction or reconstruction including bridge widening. Bridge Maintenance Improvement Program (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 BRG. Re-decking a bridge will also be handled on a case-by-case basis in consultation with BRG.

The policy does not apply to THFN overpasses, frontage roads, direct connectors not on the THFN, and entrance and exit ramps that include bridge underpasses. Direct connectors that connect THFN networks must meet the THFN vertical clearance criteria.

The THFN vertical clearance requirements should be considered for bridge and incidental vertical obstruction to the Freight Network that support significant origin/destination locations.

Refer to the **Texas Freight Advisory Committee** for more information on the development of this network plan.

21.2.1 Vertical Clearance at Structures

Vertical clearance requirements for roadway structures can be found in **Table 4-13**.

Minimum vertical clearances for bicycle and pedestrian crossover structures 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.

21.2.2 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 4-13** 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 be at least 19-ft above the pavement. Refer to the latest version of the **TMUTCD** and applicable TxDOT standards for additional guidance.

21.2.3 Other Overhead Utilities

All overhead utilities over the designated THFN project must meet the requirements specified in TxDOT's **ROW Utility Manual**, Chapter 3.

21.2.4 Bicycle and Pedestrian Accommodations

One other important consideration with freight mobility is the presence of bicyclists and pedestrians. These elements should be considered as integral parts of the THFN. See **Chapter 18** and **Chapter 19** for additional information on bicycle and pedestrian requirements. Additionally, **NCHRP Report 606** provides further information on bicycle and pedestrian considerations.

21.3 Texas Trunk System (TRUNK)

The Texas Trunk System (TRUNK) was established in 1990 by the Texas Transportation Commission to construct a network of four- lane divided rural highways. The original system was 10,050 miles including 3, 233 miles of Interstate Highway System. Another 475 miles were added in 2001. TRUNK represents 13% of state highway systems.

The purpose of this initiative was to:

- Improve rural mobility;
- Connect major activity centers;
- Provide access to ports of entry into Texas; and
- Connect with principal highways from adjacent states.

Refer to the following TxDOT reports for more information and criteria used for selection:

- **Statewide Long-Range Transportation Plan 2035**, Chapter 5: Priority Corridors (2010); and
- **Statewide and Rural Connectivity Program** (2023).

21.4 National Highway System (NHS)

The purpose of the NHS is to provide an interconnected system of principal arterial routes serving major population centers, international borders crossings, ports, airports, public transportation facilities, and other intermodal transportation facilities and major travel destinations; meet national defense requirements; and serve interstate and interregional travel.

Refer to Chapter 3 of TxDOT's **Transportation Planning and Programming Manual (TPP Manual)** for more information on the NHS.

21.4.1 History of the National Highway System

ISTEA Section 1006 called for the creation of the NHS. The National Highway System Designation Act of 1995 designates that the NHS be developed by the Secretary of Transportation in cooperation with states, local officials, and MPOs.

The NHS consists of highways designated as:

- Part of the Interstate System;
- Urban and rural principal arterials and highways (including toll facilities) which provide motor vehicle access to major ports, airports, and public and intermodal transportation facilities;
- The strategic network of highways that provides defense access, continuity, and emergency capabilities for the movement of personnel, material, and equipment in both peacetime and wartime; and
- Highway connectors which provide motor vehicle access between major military installations and highways that are part of the strategic highway network.

The first step by FHWA, State, and MPO officials was a nationwide reclassification of the Nation's roads based on their function. The target date for completion was early 1993. This allowed officials to identify all principal arterials. State transportation departments then worked with local officials to develop recommendations on which principal arterials should be included in the NHS. FHWA also worked with the DOT to identify major airport, maritime, port, rail, and transit facilities that were sufficiently important to justify connections to the NHS. A map of the system was created in 1993.

21.5 Strategic Highway Network (STRAHNET)

The STRAHNET is a 62,791-mile system across the country deemed necessary by the Department of Defense (DOD) for emergency mobilization and peacetime movement of heavy armor, fuel, ammunition, repair parts, food and other commodities to support US military operations. FHWA plays a significant role in the development, maintenance, and oversight of the STRAHNET. The FHWA works in collaboration with the Department of Defense (DoD) to ensure that the STRAHNET meets the specific transportation needs of the military while also serving broader national transportation objectives.

The Surface Deployment and Distribution Command Transportation Engineering Agency (SDDCTEA) is the agency designated for public highway matters including STRAHNET.

Below are references that provide additional information on the STRAHNET:

- **Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance** (2004); and
- **Statewide and Rural Connectivity Program** (2023).

21.6 Hurricane Evacuation Routes

The State of Texas has designated various highways and freeways as official hurricane evacuation routes to facilitate the flow of traffic during a hurricane evacuation. These routes are specifically designed to handle the increased volume of vehicles evacuating from the coastal areas affected by hurricanes.

Refer to the **Hurricane Evacuation Routes webpage** on TxDOT.gov for additional information.

21.6.1 Hurricane Evacuation Lanes

Evacuation lanes are designated emergency shoulders along hurricane evacuation routes that can be temporarily converted into travel lanes to increase capacity during an evacuation event. These shoulders may also be referred to as "EvacuLanes" and are typically wider than a traditional shoulder.

When determining the EvacuLane width, designers should consider bridges and any adjacent railings or barriers along the edge of the shoulder. **The recommended minimum width for an EvacuLane is 12-ft.**

21.6.2 Contraflow Lanes

Contraflow lanes are designated lanes that temporarily allow vehicles to travel in the opposite direction of a lane's normal traffic flow. Similar to EvacuLanes, contraflow lanes are implemented during an evacuation event when all traffic lanes move toward inland safety and away from the gulf coast. The contraflow operation roughly doubles the number of lanes available for evacuation traffic.

Chapter 22

Transit

22.1 Overview

This chapter discusses the use of transit vehicles in the design of transportation corridors. The chapter includes the following sections and subsections:

22.1 Overview

- 22.1.1 General Need;
- 22.1.2 Types of Transit;
- 22.1.3 Examples of Transit Agencies; and
- 22.1.4 Transit Accommodation.

22.2 Right of Way Requirements

- 22.2.1 Passenger Platforms;
- 22.2.2 Intersections;
- 22.2.3 Transit Centers; and
- 22.2.4 Dedicated Transitways.

22.3 Roadway Design

- 22.3.1 Lane Widths;
- 22.3.2 Profiles;
- 22.3.3 Vertical Clearances;
- 22.3.4 Acceleration and Deceleration; and
- 22.3.5 Transit Stops.

22.1.1 General Need

Highways and streets must often accommodate transit vehicles as well as passenger cars and trucks. Owners of transportation corridors, such as the State of Texas, counties, and cities are responsible for managing, funding, and coordinating the design and construction of these facilities with transit agencies. TxDOT's Public Transportation Division (PTN), as well as metropolitan transit authorities administer transit funds to ensure operation within the legal requirements in accordance with the Federal Transit Administration (FTA) regulations.

The incorporation of public transit vehicles and transit facilities requires that the roadway footprint include loading and unloading platforms, provisions for barrier-free access to and from these platforms, sufficient lighting and clear lines of sight to discourage crime. Sufficient space for additional facilities that support multi-model connections, such as shelters, ticket vending machines, shared micromobility docks, and passenger vehicle parking should also be included. These needs to accommodate public transit go beyond the basic connectivity and mobility considerations of traditional roadway design. The design of roadways that include dedicated transit corridors must be able to integrate not only the geometric needs of both existing and proposed transit vehicles, but also must include provisions in the footprint and typical section to accommodate the pedestrians, bicyclists, and micromobility users that will be travelling alongside the roadway to connect to the transit facilities. The design must encourage safety

and healthy development for all transportation users, including vehicle free facilities in the public right-of-way such as multi-use pathways.

Chapter 3 discusses contextual impacts to be considered in design and implementation of transit facilities.

22.1.2 Types of Transit

The highest capacity types of transit include steel wheel on steel rail systems such as:

- Commuter rail, usually running on shared freight lines and pulled by diesel locomotives;
- Urban subway powered by third rail;
- Urban and suburban light rail powered by overhead catenary; and
- Suburban rail powered by on-board diesel engines.

These systems generally have high speed and high capacity which can require considerable capital investment. These systems usually occupy independent corridors, but sometimes may be designed in public roadway corridors.

At the other end of transit are traditional high-capacity buses and smaller passenger vans (also referred to as microtransit) that use public roadways and are mixed into the general-purpose traffic. These often run on local and rural roads. In general, bicycle, pedestrian, and micromobility devices fill the need for first mile/last mile connections completing a transit trip. Additionally, some buses can be configured for express between two points and travel freeways, toll roads, or managed lanes. These often require minimal capital investment other than the buses, bus maintenance facilities, and furniture for bus stops.

In-between rail and buses are new innovative transit approaches such as Bus Rapid Transit (BRT). This combines the low capital cost of existing roadway assets with the efficiency and capacity of dedicated transitways. This approach was pioneered in Curitaba, Brazil as early as 1974 and included dedicated roadway for the bus vehicles, level boarding with wide doors, pre-payment at loading platforms, and articulated buses. BRT quality is ranked by the Institute for Transportation & Development Policy (ITDP) and includes criteria for engineering, planning, operations, stations, communications, and access. Full BRT systems that essentially operate as traditional rail transit are given higher scores while “BRT-light” systems that only incorporate portions of the BRT concept are given lower scores.

As technology improves, newer types of public transit may emerge. Examples might be on-demand transit using autonomous vehicles, EV transit vehicles, point-to-point autonomous vehicles designed for one or two passengers, driverless smaller vehicles on dedicated transitways with more frequent service, etc.

22.1.3 Examples of Transit Agencies

Examples of major transit agencies around Texas include major public agencies such as Dallas Area Rapid Transit, Metro (Houston), CapMetro (Austin), VIA (San Antonio), Trinity Metro (Ft Worth), and SunMetro (El Paso) as well as rural transit agencies such as McKinney Avenue Transit Authority, Galveston Island

Transit, Brazos Transit District, etc. Intercity Bus private operators such as Greyhound, Megabus, and Vonlane are often used to connect major urban areas, Each agency has a different mix of transit modes including light rail, commuter rail, bus rapid transit, on-demand, microtransit, and traditional bus. These agencies may also have their own design manuals with criteria specific to transit design which should be considered alongside the TxDOT RDM. If the criteria conflict, it should be resolved prior to any significant design activity taking place.

22.1.4 Transit Mix

Design of roadways to accommodate transit vehicles can include traditional mixed traffic and curbside/shoulder bus stops, mixed traffic with protected areas for passenger loading/unloading, mixed traffic with priority to transit vehicles, dedicated at-grade guideways within the roadway section, and dedicated at-grade and elevated guideways using an independent corridor. A transit corridor may even contain a mix of these modes within the same corridor depending on the available ROW and cost/benefit analysis of the transit service.

22.1.5 Transit Accommodation

Design of the roadway section for transit vehicles includes areas beyond the pavement limits, such as adequate safety buffers between roadway and sidewalks, space for trees outside the clear zone behind curb to shade sidewalks, areas where bike sharing docks or microbility parking can be installed, space for shelter, information, and ticket vending, etc. It is important for roadway design engineers to coordinate with other urban design disciplines to avoid precluding current and future transit needs with incompatible geometry or roadway section. Transit design should also accommodate Vulnerable Road Users (VRUs).

22.2 Right of Way Requirements

22.2.1 Passenger Platforms

The distance from the edge of the traveled way to the ROW line may need to be increased to allow adequate passenger platforms to be installed. In areas where there are either dedicated bikeways or shared use paths, the through bicycle and/or pedestrian traffic may need to be routed behind the passenger platform on additional ROW to avoid interference with passenger waiting or boarding. **As a general policy, TxDOT does not purchase ROW for Transit or Bus Stops.**

22.2.2 Intersections

Additional ROW may be required around intersections with cross streets, light rail tracks, and railroad tracks to allow adequate bike facility geometry. Two-wheeled vehicles need to cross tracks and other linear obstacles in a roadway as close as practicable to a right angle. Additional room at corners may be needed not only for sight distance around passenger platforms, but also for transit staging areas or bike sharing docks.

22.2.3 Transit Centers

New ROW outside of the normal roadway limits may be required for transit centers that allow passengers to transfer vehicles without waiting on the side of the street. These transit centers may require special geometry to allow buses to efficiently enter and exit without increasing the risk of incidents with general purpose traffic. These transit centers also often provide bicycle/micromobility parking, and private vehicle access facilities like parking, park-and-ride, and drop off facilities for public toilets, information kiosks, ticket vending machines, transit police, and climate controlled spaces for extremely hot or cold weather.

22.2.4 Dedicated Transitways

Dedicated transitways for Bus Rapid Transit (BRT), in-street Light Rail Transit (LRT), and Streetcars (e.g., Trams or Trolleys) allow transit vehicles to travel at the design speed regardless of the congestion on the general-purpose lanes. However, ROW constraints may require the reconfiguration of the existing typical section, such as the narrowing of a wide median or combining transit and right turn lanes into a Business Access Transit (BAT) lane. These approaches can accommodate transit without reducing existing capacity. It may still be necessary to acquire additional ROW at signalized intersections to provide storage for left turn lanes. Locating transit platforms away from turn lanes can help to mitigate additional ROW needs.

Note that if existing medians are utilized for dedicated transitways, existing unsignalized intersections that allow left turns from and to the major roadway will become “T” intersections with right turns only. Cross traffic will only be possible at signalized intersections.

If existing rail corridors are retired and the ROW is returned to the government, these corridors may be repurposed as dedicated transitways with or without parallel multi use bike/pedestrian trails.

22.3 Roadway Design

22.3.1 Lane Widths

Lanes that carry transit vehicles require greater width than passenger vehicle traffic. Lanes should optimally be at least 12-ft wide to accommodate not only the width of a transit vehicle but also the width of the rear-view mirrors. It is recommended that the lane widths be no less than 11-ft.

22.3.2 Profiles

Because transit vehicles often have long wheelbases and long front and rear overhangs, sharp changes in profile or vertical curves with low K values should be avoided. Profiles across intersecting streets and more critically across railroads should be as flat as possible or have high K values to avoid any possibility of the bottom of the transit vehicle contacting the pavement.

Limiting grades for transit vehicles depend on the power source. For diesel powered vehicles short upgrades of up to 6% are acceptable and short ramps may be up to 10%. Electrically powered vehicles often have greater hill-climbing ability than diesel powered vehicles.

Transit stations and other developed stops should ideally be on profiles of 1.5% or less to allow for accessible boarding. However, retrofits on existing roadways or profiles in hilly terrain may make this requirement infeasible. In those cases, **PROWAG** allows for exceptions.

Low points in profiles should not be designed where a transit platform or bus pullouts are planned as the presence of a platform makes an inlet impractical and creates drainage issues.

22.3.3 Vertical Clearances

Most transit vehicles require at least 14.5-ft of vertical clearance, not just for the vehicle height but also to accommodate equipment on top of the vehicle such as air conditioners and for future resurfacing. Transit vehicles that are fed electrical power from the top may require greater clearances depending on the design of the transit vehicle and catenary electric system.

22.3.4 Acceleration and Deceleration

Corridors with transit vehicles should be designed assuming lower acceleration and deceleration rates than passenger vehicles. Normal acceleration should be estimated at 3 ft/s^2 and normal deceleration should be estimated at 2 ft/s^2. When vehicles with standing passengers use a corridor, the design should limit maximum deceleration to 8 ft/s^2. This may significantly affect the yellow times for signalized intersections.

Note that while the maximum deceleration for transit vehicles is less than the 11.2ft/s^2 normally assumed for stopping sight distance for passenger vehicles, the driver's eye height of a transit vehicle is usually significantly higher than the 3.5-ft used for passenger vehicles, allowing for greater sight distance over crest vertical curves. For sag vertical curves the headlights on transit vehicles are usually mounted higher than passenger vehicles, which should provide for greater sight distance at night. Note that increased driver eye

height/headlight height does not increase sight distance around horizontal obstructions, so transit-heavy corridors may need to be evaluated with larger sight triangles that might be normally used for passenger vehicles.

22.3.5 Transit Stops

Transit stops may be located in the center of a roadway between dedicated transit lanes with left side boarding, between dedicated transit lanes and general-purpose lanes with right side boarding, or in outside general purpose or BAT lanes with right side boarding.

Outside lane transit stops, or transit lines without transit signal priority on the signalized intersections, should usually be located on the far side of a signalized intersection so that the transit vehicle can pass through the intersection on a green before needing to stop to pick up and discharge passengers. This allows for passing traffic to continue and right turns to flow.

While transit vehicle pull-outs at stops may allow through traffic to pass while the transit vehicle is stopped discharging and picking up riders, it also may make it difficult for the transit vehicle to re-join the traffic stream when traffic density is high. In these cases, it may be necessary to design the signal timing in such a way that the transit vehicle is provided sufficient gap to leave the pullout and re-enter the general-purpose lane.

Transit stop height may vary from normal curb height (usually 6-in) to level boarding (usually 15-in to 18-in) or somewhere in between with "near-level boarding". Transit stops without level boarding may require transit vehicles to use vehicle-mounted technology to accommodate wheelchairs. A non-vehicle alternative is for the platform to have a level boarding "hump" which lines up with the front door. This requires that the transit vehicle operator stop at an exact location at the boarding platform. These humps should have slopes of 5% or less to conform to ADA requirements and may require guard rails in the back for safety.

In some cases, BRT transit stops may need to be designed adjacent to existing roadways with longitudinal grades near or exceeding the 5% ADA maximum. In these cases, while PROWAG provides an exception to ADA for running slopes, care should be taken in the design of transitions from normal curb height to the height of the transit platform. These designs should be discussed with the assigned Registered Accessibility Specialist (RAS) before plans are submitted for review to clarify PROWAG requirements. This will help avoid any post-construction misunderstanding about conformance to Texas Accessibility Standards.

Although transit planning generally prefers locating major transit stops adjacent to signalized intersections to allow signal-protected crossings, ROW and commercial driveway constraints may make this impractical. If mid-block transit stops are considered, the designer should consider including crossings protected by pedestrian hybrid beacons so that pedestrians do not need to walk all the way back to the nearest signalized intersection to cross to the other

side of the roadway. Specific requirements for the application of pedestrian hybrid beacons are addressed in **Chapter 19.**

Bicycle and through pedestrian traffic on sidewalks/sidepaths should ideally be routed around the back of platforms that are located on the outside lane to avoid interfering with riders waiting and boarding/deboarding. However, in cases where this is not feasible, platforms should be designed so riders may wait in areas away from through bike/ped traffic and through bike/ped traffic is advised to stop when transit vehicles are boarding/deboarding.

Profiles through transit stops should ideally be designed at no more than 1.5% to allow for wheelchairs to turn in any direction to board/deboard vehicles. However, if the transit stop is controlled by the profile of the adjacent roadway, this might not always be practical.

Transit stops should be designed on horizontal tangents of roadway to minimize the gap between the BRT vehicle and the passenger platform. Horizontally curved platforms can be difficult for BRT vehicles to correctly dock against.

Chapter 23 Temporary Traffic Control Design

23.1 Overview

This chapter discusses the use of Temporary Traffic Control during roadway construction and provides general design considerations. The chapter includes the following sections and subsections:

23.1 Overview

23.2 General Standards and Practice

- 23.2.1 Purpose of a Traffic Control Plan (TCP) Design;
- 23.2.2 Safety in TCP Design; and
- 23.2.3 Federal Regulations relevant to TCP Design.

23.3 TCP Design Considerations

- 23.3.1 Duration;
- 23.3.2 Cross Sectional Elements;
- 23.3.3 Pedestrian and Bicycle Requirements:
- 23.3.4 Property Access Requirements;
- 23.3.5 Temporary Geometry Requirements;
- 23.3.6 Temporary Pavement Structure Requirements;
- 23.3.7 Temporary Drainage Requirements;
- 23.3.8 Temporary Clearance Requirements;
- 23.3.9 Temporary Ramp Design;
- 23.3.10 Business Access Requirements; and
- 23.3.11 Emergency Vehicle Access Requirements.
- 23.3.12 Treatment of Pavement Drop-offs in Work Zones

23.2 General Standards and Practices

23.2.1 Purpose of a Traffic Control Plan (TCP) Design

Where the normal operation of a roadway is suspended, a TCP design, sometimes referred to as a Temporary Traffic Control (TTC), will provide for the continuity of vehicle, bicycle, and pedestrian traffic as well as access to property and utilities. There are two objectives of a TCP design:

1. Provide for reasonably safe and efficient road-user movement through or around a roadway work zone to protect workers, incident responders, and equipment.
2. Provide for the efficient completion of the construction or maintenance activities interrupting the normal operation of the roadway.

23.2.2 Safety in TCP Design

The primary goal of TCP design is to provide for safe movement of vehicles and other users through the construction zone as well as the protection of construction workers from those vehicles. While a certain amount of risk is inherent in any design as it is balanced against construction cost, mobility, and access, a TCP may have greater risk due to constantly changing alignments which may concern drivers who have experience navigating through that work zone, movement and operation of construction equipment and personnel, and TCP devices that need contractor maintenance.

23.2.3 Federal Regulations relevant to TCP Design

The Federal Highway Administration updates (released September 9, 2004) to the work zone safety regulations under **23 CFR 630 Subpart J** are referred to as the “Final Rule on Work Zone Safety and Mobility”.

The Rule updates and broadens the former regulation to address more of the current issues affecting work zone safety and mobility impacts of the work zones across project development and the implementation of strategies that help managed these impacts during project delivery.

The updated rule applies to all State and local governments that receive Federal-aid highway funding. All of these agencies were required to comply with the provisions of the rule no later than October 12, 2007.

In accordance with **Subpart J and K**, TxDOT released the Work Zone Safety and Mobility Guide, most recently updated in November 2023. This guidance document contains strategies for work zone safety and mobility management, including requirements from the **TMUTCD** as well as TxDOT standard sheets and manuals. In accordance with **23 CFR 630.1006**, this document supports TxDOT's policy for the systematic consideration and management of work zone impacts and incorporates input from a multi-disciplinary team that includes FHWA and other external partners

The designer of a TCP must have a thorough understanding of common safety risks in transportation construction in order to produce an effective plan.

23.2.3.1 Transportation Management Plan

A key requirement in the Federal Regulation is the development and inclusion of a Transportation Management Plan (TMP) as part of the project development and contract administration processes.

A TMP (detailed in **23 CFR 630.1012(b)** is an overall strategy to accommodate traffic during road work that minimizes adverse impacts and maximizes safety and/or mobility, as well as vehicular delay, in the general area affected by the project. The scope, content, and degree of detail present in a TMP will vary based on identifying the project. Significant projects, as defined in **23 CFR 630.1010**, may not only require a TCP design for the project, but regionally-focused Transportation Operations (TO) and Public Information (PI) plans. However, an effective TMP should typically include:

- Project Description;
- Existing Traffic & Roadway Conditions;
- Proposed Work Zone Strategies;
- Potential Work Zone Impacts;
- Alternative Work Zone Strategies;
- Referenced Work Zone Documents; and
- Construction Monitoring.

23.2.3.2 Standard Devices

Part 6 of the **TMUTCD** and the **TxDOT Traffic Standards – Traffic Control Plan** provide guidance for standard devices used for temporary traffic control, both short term and long term. These standard devices should be chosen based on site-specific engineering judgement.

23.3 TCP Design Considerations

23.3.1 Duration

Work durations are defined by the TMUTCD and by the **TxDOT Traffic Control Plan Selection Worksheet**. TCP of 3 days or less may be implemented by the contractor using the **TxDOT Control Plan Standards**. An engineered (signed & sealed) traffic control design shall be required and provided to the contractor in the construction plans for Long Term Stationary work that is for an expressway/freeway. Unique project conditions, such as detours of major arterials or rapid bridge replacement involving higher risk to the travelling public, may justify TCP plans of 3 days or less to be designed as part of the construction plans.

23.3.2 Cross Sectional Elements

23.3.2.1 Typical Sections

Typical sections for TCP are generally much more restrictive than permanent typical sections. Cross slopes may vary greatly depending on the nature of the existing, temporary, or permanent pavement upon which the traffic is being maneuvered. Lane and shoulder widths are allowed to decrease and edge conditions are allowed to be more aggressive before protection by concrete barrier is required, as described below.

23.3.2.2 Lane Widths

Typically, lane widths for TCP plans should not be less than one foot narrower than the lane widths for the permanent design. However, if the permanent lane width is already under a design exception, the TCP lane width should not be decreased any further. Alternatively, in cases where the desired minimum lane width is not feasible, for instance to temporarily pass traffic through a narrow obstruction, a narrower lane width may be considered using engineering judgement, provided the alignment is straight through the entire approach and departure of the obstructions. In these extreme cases, lanes should be striped with a continuous solid white line to prohibit lane changes.

23.3.2.3 Shoulder Widths

Typically, a shoulder width of two feet or greater is desirable to reduce the probability that traffic control devices or concrete barrier will be struck, as well as to minimize the chance that debris collected against a concrete barrier will obscure the temporary edge striping. However, in extreme cases the shoulder width could be reduced to zero if there are no other options.

In areas where traffic is constrained by concrete barrier for a considerable distance, safety pullouts for disabled vehicles should be provided at regular intervals based on engineering judgement. These areas should be clearly signed that they are for emergencies only.

23.3.3 Pedestrian and Bicycle Requirements

According to the **TMUTCD** Section 6A.01, road users include not only motorists, but bicyclists, pedestrians, and persons with disabilities in accordance with the **Americans**

with Disabilities Act of 1990 (ADA), Title II, Paragraph 35.130.

The requirements in the **TMUTCD** are compulsory, therefore, a TCP design should consider the needs and control of these users in every phase of construction. Safe and effective paths should be maintained not only for instances where pedestrian or bicycle facilities currently exist, but also at locations where accommodations may not currently exist but there is known use by pedestrians or bicyclists.

The designer should perform an assessment of the existing facilities for pedestrians and cyclists – including widths, grades, surface conditions, pavement markings, signing, anticipated usage, and access to these facilities. The designer should document the impact assessment in the DSR.

23.3.3.1 Pedestrians

If the TCP zone affects the movement of pedestrians, the designer should provide for adequate pedestrian access and walkways. If the temporary traffic control zone affects an accessible and detectable pedestrian facility, the accessibility and detectability shall be maintained along the alternate pedestrian route (**TMUTCD**, Section 6D.01 Pedestrian Considerations, 04). Refer to the **TMUTCD** for additional information on accommodations for the visually impaired.

In accommodating pedestrians, the following principles should be applied, addressed, and incorporated into the TCP design:

- Do not lead pedestrians into hazardous environments or conflicts with public traffic, construction vehicles, equipment, or operations.
- Use existing intersection corners and crosswalks when directing pedestrians across a roadway. An existing marked mid-block crossing may be used to shorten pedestrian routes.
- Provide a convenient, contiguous pathway that is equivalent to the existing level of pedestrian accessibility.
- Minimize out-of-direction travel for pedestrians.
- Sign the closure of a pedestrian route in a minimum of two locations:
    - In advance of the closure point at the nearest alternate crossing or diversion point; and
    - At the closure point itself.
- Avoid having a pedestrian route double-back on itself. Pedestrians are not likely to walk one block beyond the closure to the next crossing and then one block back on the other side of the road. They will likely cross before the work zone impact (if visible), or midblock – which may be unsafe or leave the pedestrian within the work area.
    - Confirm whether visually impaired pedestrians can be expected in the work zone. Techniques include:

- Personal investigations and/or collecting manual counts; and
- Coordinate with local agency/organization sources, and other stakeholders.

23.3.3.2 Bicycles

Designers should investigate existing conditions within the limits of their project to determine what accommodations, if any, need to be established for bicycles. Some of these conditions include:

Closure of Existing Shoulders, Sidewalks, Bike Lanes or Shared-Use Paths: The TCP design should provide temporary facilities or pathways that allow for safe, efficient bicycle travel through or around the work zone. Long term TCP designs often leave minimal operational widths for vehicles with little to no shoulder in a corridor that may commonly be used by bicycles even if there is no dedicated bicycle lane. Consider alternative bicycle routes when 4-ft or more of pavement outside of the vehicular lane is not feasible. Shared use of the roadway by both motorists and bicyclists may be considered, but it generally not recommended, especially for high-speed traffic.

Urban/Urban Core/Suburban/Rural Town Intersections: The scope of work often includes all four corners of an intersection and can intrude into the shoulder/bike lane. In some of these environments, viable detours or space for adequate temporary facilities may be limited. If measures can be deployed to ensure a speed reduced to 35 or below through the work zone, use of a shared roadway condition may be a measure for accommodating bicyclists.

Construction Details: Where applicable, TCP Plans must provide enough detail to safely accommodate bicycle traffic within the work zone. Bicycle-specific details may be included on separate plan sheets, depending on the level of complexity at each location where special bicycle channelization is needed. Include details such as:

- Bicycle traffic routes;
- Bicycle channelizing devices;
- Bicycle-specific temporary signs;
- Cross sections at critical locations or "pinch points" where precise placement of traffic control devices (TCD) is vital; and
- Temporary surfacing material and markings.

Occasionally neither on-site roadway widths nor local detours are available. In these cases, innovative means of transporting bicycles through the construction area should be considered and weighed against traditional measures. Partnerships with public transit or private shuttle services have been used in the past to maintain acceptable levels of bicycle mobility. Consider temporary bus/shuttle stops, information kiosks, "hotline" phone numbers, etc., to provide an effective transportation means for cyclists.

23.3.4 Property Access Requirements

Construction at property access points, such as residential and commercial driveways, that do not have any alternate access points away from the construction, should be staged so that the property owner always has the ability to enter and leave their property by vehicle. The designer should provide the contractor with details for how to stage construction at these access points, for instance doing half a side at a time, even if it reduces the width to the point where only one vehicle can pass through at a time.

If maintaining access to a property during construction is extremely problematic then the contractor will need to coordinate with the property owner to implement an agreed plan in writing for offsite parking, pedestrian access, service and fire vehicle access, or total temporary access denial.

23.3.5 Temporary Geometry Requirements

The geometric alignment of the TCP should be similar to the geometric alignment of permanent design. Section 6B.01 of the **TMUTCD** states: "The basic safety principles governing the design of permanent roadways and roadsides should also govern the design of TCP zones. The goal should be to route road users through such zones using roadway geometrics, roadside features, and TCP devices as nearly as possible comparable to those for normal highway situations."

23.3.5.1 Horizontal Geometry

Horizontal geometrics for short duration, short term stationary, and intermediate term stationary traffic control alignments may be implemented using the diagrams in the **TMUTCD** and **TxDOT Traffic Standards – Traffic Control Plan**. These implementations are not likely to involve the temporary detour pavement, pavement markings, or concrete barriers that are often required for long term stationary alignments.

For long term stationary alignments, lateral shifts in the traffic not more than one lane width in distance may be designed using shift tapers. However, for traffic shifts greater than one lane width, or for complex alignment, designed horizontal curvature is preferable.

TCP horizontal curves should be designed so that the side forces on the vehicles do not exceed the recommended maximum safe side forces for various design speeds (refer to the Maximum f column of Table 3-7 of the 2018 **AASHTO A Policy on Geometric Design of Highways and Streets**).

The TCP design may require a shift of alignment of traffic in locations other than straight, normally-crowned roadway. The designer should consider the cross slope of the pavement upon which the curves are designed, and whether or not the cross slope is towards or against the direction of the curvature. Minimum radii for combinations of TCP design speed and cross slope are shown in **Table 23-1** below.

Table 23-1: Minimum Radii for Horizontal Curvature of TCP Geometry on Existing or Temporary Pavement

TCP Design Speed									
Cross Slope	f max 20%	f max 18%	f max 16%	f max 15%	f max 14%	f max 13%	f max 12%	f max 11%	f max 10%
	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)
-8%	500	817	1,334	1,929	2,778	4,034	6,000	9,389	16,334
-7%	462	743	1,186	1,688	2,381	3,362	4,800	7,042	10,889
-6%	429	681	1,067	1,500	2,084	2,881	4,000	5,634	8,167
-5%	400	629	970	1,350	1,852	2,521	3,429	4,695	6,534
-4%	375	584	889	1,228	1,667	2,241	3,000	4,024	5,445
-3%	353	545	821	1,125	1,516	2,017	2,667	3,521	4,667
-2%	334	511	762	1,039	1,389	1,834	2,400	3,130	4,084
-1%	316	481	712	965	1,283	1,681	2,182	2,817	3,630
0%	300	454	667	900	1,191	1,552	2,000	2,561	3,267
1%	286	430	628	844	1,112	1,441	1,847	2,348	2,970
2%	273	409	593	795	1,042	1,345	1,715	2,167	2,723
3%	261	389	562	750	981	1,261	1,600	2012	2,513
4%	250	372	534	711	926	1,187	1,500	1,878	2,334
5%	240	356	508	675	878	1,121	1,412	1,761	2,178
6%	231	341	485	643	834	1,062	1,334	1,657	2,042
7%	223	327	464	614	794	1,009	1,264	1,565	1,922
8%	215	315	445	587	758	961	1,200	1,483	1,815

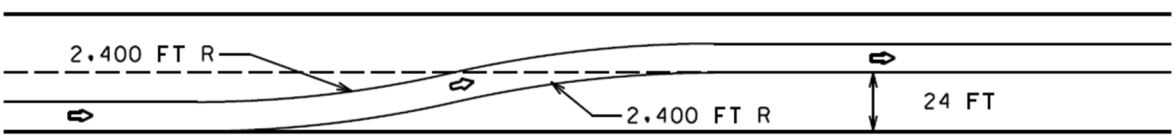
Note that these TCP radii match the radii found in Table 3-13 Minimum Radii and Superelevation for Low-Speed Streets in Urban Areas, **AASHTO A Policy of Geometric Design of Highways and Streets (2018)**, which follows Superelevation Distribution Method 2. However, this table has been developed to include higher TCP design speeds across a higher range of existing cross slopes using the formula $V^2/15R = e + f$. The designer also has the option of using a more conservative superelevation methodology (see **Table 4-3**). **Table 23-1** should not be used for permanent design of high-speed facilities. Permanent conditions must use the superelevation methodology indicated in **Table 4-3**).

23.3.5.2 Design Examples:

23.3.5.2.1 Example 1:

The TCP designer needs to lay out a long-term shift of traffic across a 2% normal crown where the work zone design speed is 60 mph and the traffic is being shifted laterally by 24-ft. A shift taper based on WS/2 would require 720-ft with a 1.9 degree diverging angle, while a reverse curve using a 2,400-ft radius could be accomplished in 479-ft. In this case the added complexity of laying out a reverse curve in construction may be justified in order to save nearly 250-ft of transition distance as well as resulting in smoother navigation of vehicles through the lateral shift.

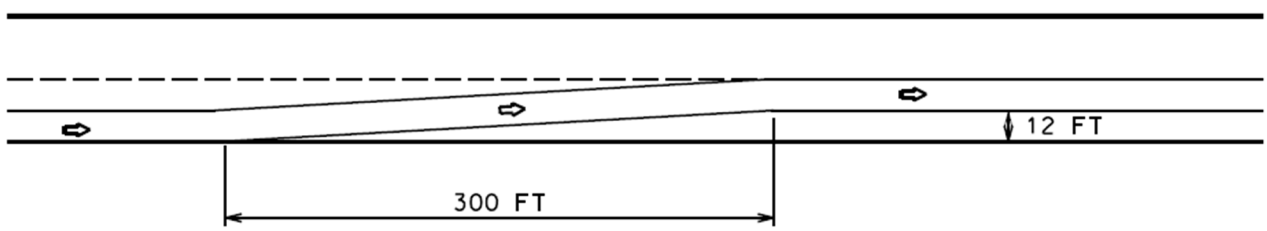
- Long term shift
- Work zone design speed = 60 mph
- Existing roadway cross slope = -2%
- Shift width = 24-ft



23.3.5.2.2 Example 2:

A TCP designer needs to lay out a long-term shift of traffic across a 2% normal crown where the work zone design speed is 50 mph and the traffic is being shifted laterally by 12-ft. A shift taper based on WS/2 should be adequate in this case resulting in a diverging angle of 2.3 degrees and a shift length of 300-ft, through which a vehicle may naturally spiral within the width of the lane as it navigates. Although a reverse curve using a 1,389-ft radius may be smoother and produce a slightly shorter shift length of 258-ft, the distance saved is probably not worth the additional complexity required for construction layout.

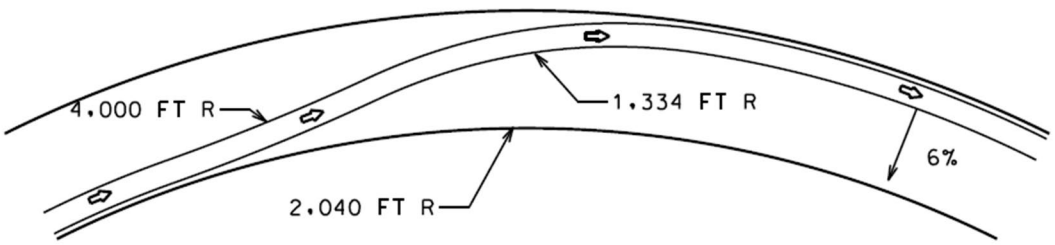
- Long term shift
- Work zone design speed = 50 mph
- Existing roadway cross slope = -2%
- Shift width = 12-ft



23.3.5.2.3 Example 3:

The TCP designer needs to lay out a long-term shift of traffic on existing or new pavement through a horizontal curve with a 6% superelevation where the work zone design speed is 60 mph. In this case the vehicle path could be curved opposite the direction of the the cross slope with a radius of 4,000-ft without exceeding the maximum side force. A tangent should be avoided as this might cause the driver to speed up. The vehicle path could be brought back into the horizontal curve with a 1,334-ft radius in the direction of superelevation slope. This would accomplish the shift in the minimum possible distance.

- Long term shift
- Work zone design speed = 60 mph
- Existing roadway cross slope = -6%



23.3.5.3 Vertical Geometry

The K values of sag and crest vertical curves in a TCP design should use the same requirements as permanent design as specified in **Chapter 4**. If these K values cannot be reasonably achieved, the designer should consider a further reduction in the work zone speed limit for the specific construction phase or location involved to shorten the stopping sight distance. This reduced speed should be clearly posted as an advisory sign.

23.3.6 Temporary Pavement Structure Requirements

Temporary detours and widening required for a TCP design should follow the guidance shown in TxDOT's **Pavement Manual**, Section 5. Prior to designing an alignment using temporary pavement, the designer should perform a cost analysis between the use of temporary pavement (including any temporary drainage, embankment, and structures required) and alternate construction phasing that takes advantage of using either existing or permanent pavement.

23.3.7 Temporary Drainage Requirements

Stormwater drainage for temporary conditions during construction where the permanent drainage has not yet been constructed or connected should be designed using the guidance in TxDOT's **HDM**. The expected capacity for which the temporary system should be designed will depend on the expected duration of the temporary condition. A conservative approach for typical construction phases is to assume a rainfall event with a 50% Annual Exceedance Probability, or a 2-year Annual Recurrence Interval.

Temporary drainage should be designed using positive drainage in an TCP phases and should not assume the contractor will use pumps to drain low points. Design of the temporary drainage should be coordinated with the design of the Storm Water Pollution Prevention Plan (SWP3).

Refer to TxDOT's **HDM**, Chapter 8, Section 6 – Special Applications for discussion on risk assessment during construction.

23.3.8 Temporary Clearance Requirements

23.3.8.1 Clear Width for Controlled-Access Highways

A TCP design should be of sufficient width so that oversize vehicles permitted to operate on Texas highways can clear any obstructions by at least one foot, particularly where there are concrete barriers on both sides or when the alignment has a small radius curve. As the maximum width allowed by Texas permit on controlled access highways is 16-ft, clear width should be no less than 18-ft in one direction. As the maximum width allowed by Texas permit without a certified route inspection is 20-ft, any clear width less than 22-ft should be mitigated. For highways other than controlled access, horizontal clearance should not be reduced to less than the existing roadway width, excluding marked shoulders, unless restrictions are posted and detours provided for loads exceeding that width.

If achieving minimum clear width is not possible, the maximum width allowed should be posted in advance of the work zone and a detour for oversize vehicles must be provided.

Lane Widths: 10-ft lane widths are permissible for freeways in straight alignments where truck off-tracking is not likely but should only be used in limited circumstances where wider lanes are not feasible.

Shoulders: When improved shoulders are closed due to construction constraints, road users should be given ample warning. (**TMUTCD** Section 6G.07). Periodic pullouts for refuge should be considered if the construction phasing will reasonably allow.

Clear Zone: Where traffic patterns are shifted into an existing clear zone, it is possible that objects that were previously outside of the clear zone will increase risk to the relocated traffic (culvert headwalls, ditch inlets not rated for traffic, side slopes greater than 6:1, etc). Consideration should be given to installing temporary traffic barrier if these hazards cannot be otherwise mitigated.

Lateral Clearance and Barrier Placement: For operational and safety reasons, a minimum of 1-ft lateral offset, 2-ft preferred, should be provided from traffic control devices, especially concrete barriers, to the edge of traveled way during all phases of construction. This becomes more important the narrower the lanes are, the higher the ADT is, or the higher the truck percentage is.

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.

Where TCDs, especially concrete barrier, are placed downstream of an entrance ramp lacking an acceleration lane, provide as much space as practicable at the entrance ramp merge with the freeway, allowing the traffic to encroach over the striping as needed to avoid collisions.

Where concrete barrier is placed upstream of an entrance ramp, ensure that sight distance is not obstructed for traffic making the merge. Likewise, for exit traffic merging with frontage road traffic

23.3.8.2 Clear Width for Non-Access Controlled Roadways

A lane width of 9-ft may be used for short-term stationary work on low volume, low speed roadways when vehicular traffic doesn't include longer and wider heavy commercial vehicles (**TMUTCD** Section 6G.08).

23.3.8.3 Considerations for Texas Highway Freight Network (THFN) or Equivalent Routes

The following should be considered when a TCP is conducted on the THFN:

- At least one 12-ft lane is encouraged. Use wider outside lane than inside lane if lane widths are restricted.
- Use longer than minimum lane closure and lane transition lengths.
- Use solid striping to discourage weaving through significant alignment shifts.

Use portable changeable message signs (PCMS) and other warning signs far in advance of work when lane closures or significant changes occur.

23.3.8.4 Vertical Clearance on Controlled Access Highways

Refer to TxDOT's **Bridge Design Manual - LRFD** for considerations associated with temporary vertical clearances during construction. Discuss with the Project Manager when temporary vertical clearances may be a controlling condition for roadway geometry.

At a minimum, bridge vertical clearances above all the travelable pavement must be a minimum of 14'-6" (15'-0" preferred) during construction to provide a sufficient buffer between allowed and actual heights to avoid over-height impacts.

Any temporary vertical clearances that are less than existing vertical clearances upstream from the construction work zone should be clearly posted and a detour provided, as necessary, for vehicles that would normally have been able to safely pass under the pre-existing structures. It is also recommended that either an automated height measurement sensor or a physical clearance bar should be installed to alert operators of any vehicles who fail to take the detour that they are over the temporary height limit.

23.3.9 Temporary Ramp Design

For entrance and exit ramps in work zones, the TCP design should allow as much space as practicable for decelerating and accelerating vehicles, particularly where heavy truck volumes exist. Include acceleration and deceleration lanes where warranted and feasible. Where parallel acceleration and deceleration lanes are not feasible to provide, use as large of an exit radius as construction constraints will allow, thereby avoiding excessive slowing in the main lanes for exits, and as long of an entrance taper as construction constraints will allow to minimize the disparity of speed for entrance ramp traffic compared to the freeway traffic speed.

For full reconstruction where the new roadway profile differs from the existing roadway profile, temporary entrance and exit ramps may need to be designed at those locations where the existing and proposed profiles cross to minimize elevation differences. This approach helps to simplify the construction phasing and reduce associated phasing cost. If this is not possible, additional horizontal clearance during construction will be needed to transition the elevation of the temporary ramp from one pavement surface to the other, with sufficient length to accommodate vertical geometry

requirements. If all feasible ramp design options have been exhausted, it may be necessary to temporarily close ramp access and provide detours for traffic that needs to enter and exit the freeway.

23.3.10 Business Access Requirements

A TCP design should include traffic control measures to help delineate business accesses disrupted by construction. Driveway approaches for private businesses are occasionally disturbed making them less visible to passing traffic and particularly difficult to find at night or during inclement weather.

In an effort to partner with local businesses affected by construction, the engineer may specify additional signing and special channelizing devices to clearly identify temporary business accesses. Business access signs consistent with the **TMUTCD** should be coordinated in the construction public involvement plan and included in the construction documents for the contractor.

23.3.11 Emergency Vehicle Access Requirements

A TCP design should address the accommodation of emergency vehicles through or around the construction area. For emergency vehicles that would normally pass through the work zone, alternate routes should be planned and communicated to all local emergency responders as an option should the work zone be congested or otherwise impassable. For emergencies within the work zone, the long-term design should allow access from the downstream side of the emergency, or in the opposing lanes once law enforcement has cleared that traffic.

23.3.12 Treatment of Pavement Drop-offs in Work Zones

The Guidelines depicted in the Traffic Safety Division EDGECON Standard Detail apply to construction zone work where continuous pavement drop-offs exist parallel and adjacent to a lane used for travel. The guidelines do not apply to short-term operations, and do not constitute a rigid standard or policy; rather they are guidance to be used in conjunction with engineering judgement.

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.

Additionally, 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 motorcyclists 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 23-1**.

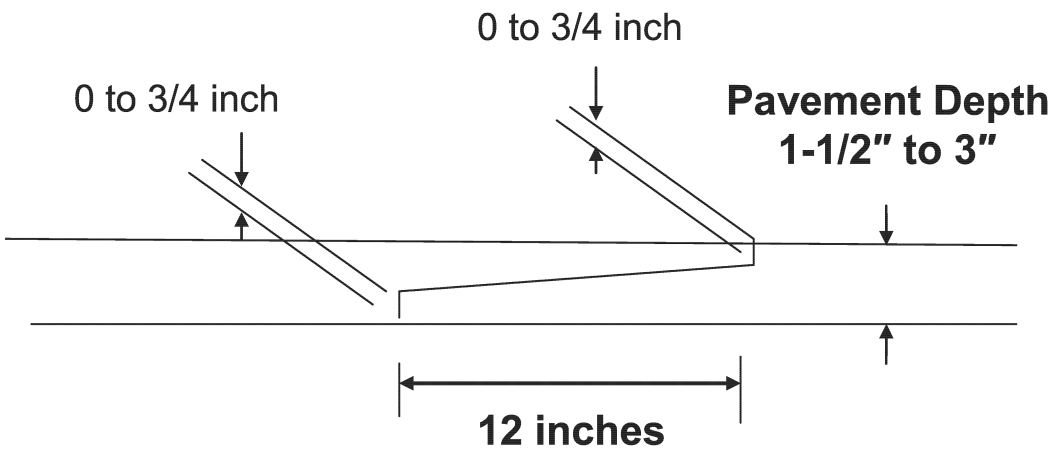


Figure 23-1: Notched Wedge Joint Detail

Refer to the Construction Divisions technical advisory report on **Use of Tapered Longitudinal Joints such as the Notched Wedge Joint** for additional information.

Chapter 24

Other Design Elements

24.1 Overview

This chapter discusses other design elements to be considered during roadway design. The chapter includes the following sections and subsections:

24.1 Overview

24.2 Fencing

- 24.2.1 Overview;
- 24.2.2 Property Fencing on ROW; and
- 24.2.3 Control of Access Fencing on Freeways.

24.3 Illumination

- 24.3.1 Overview;
- 24.3.2 General Considerations; and
- 24.3.3 Other Safety Benefits.

24.4 Signing

- 24.4.1 Overview;
- 24.4.2 General Considerations; and
- 24.3.3 Other Safety Benefits.

24.5 Pavement Markings

- 24.4.1 Overview;
- 24.4.2 General Considerations; and
- 24.3.3 Other Safety Benefits.

24.6 Intelligent Transportation Systems (ITS)

- 24.4.1 Overview;
- 24.4.2 General Considerations; and
- 24.3.3 Other Safety Benefits.

24.7 Wildlife Crossings

- 24.7.1 Overview;
- 24.7.2 Background;
- 24.7.3 Overpass Crossings;
- 24.7.4 Underpass Crossings;
- 24.7.5 Culvert Crossings;
- 24.7.6 Barriers;
- 24.7.7 Wildlife Guards; and
- 24.7.8 Fencing and Signing.

24.8 Texas Aggregate Quarry and Pit Safety Act

24.2 Fencing

24.2.1 Overview

This section discusses the features and design criteria for fencing and includes the following subsections:

- ROW; and
- Control of Access Fencing on Freeway

24.2.2 Property Fencing on ROW

Procedures for fencing highway ROW are in Chapter 8 of TxDOT's **ROW Acquisition Manual**. Where additional ROW is not required for construction of improvements of existing highways, ROW (property) fencing is the responsibility of the landowner. Where additional ROW is required, the replacement or reestablishment of privately owned property fences along the ROW is handled as a cost to cure item. Construction, maintenance and ownership of the fence is vested with the landowner.

24.2.3 Control of Access Fencing on Freeway

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 24-1** 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 where other special conditions exist. Control of Access fencing is constructed and maintained by the State.

Table 24-1: Fence Types for Various Conditions

Condition	Type of Fence	Usual Location
Urban and suburban areas	Chain link fence of 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-2-in	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 halfway 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 ROW line, the control of access fence may be located at the ROW line and will serve a dual function as a ROW fence.

24.3 Illumination

24.3.1 Overview

Roadway illumination allows motorists and pedestrians to identify potential risks which might otherwise be difficult to detect on an unlit portion of roadway. Ample lighting should be provided for the safety of both motorists and pedestrians. The type of lighting required should be determined and specified in the design process. The two basic types of roadway lighting systems are continuous lighting and safety lighting:

- Continuous Lighting is a system that provides relatively uniform lighting on main lanes, direct connections, ramp terminals, and complete interchanges. Frontage roads are not normally continuously illuminated by TxDOT.
- Safety Lighting is typically installed at an interchange, highway intersection, rural freeway ramps or other decision-making point or points of nighttime risks. Safety lighting may be used to the extent necessary to provide for safety enhancement and the orderly movement of traffic.

Refer to TxDOT's **Highway Illumination Manual**, the **AASHTO Roadway Lighting Design Guide**, **TMUTCD**, TxDOT's **Landscape and Aesthetics Design Manual** Chapter 4 Section 9, **ANSI RP-8**, **FHWA Lighting Handbook**, and TxDOT standards for additional information and guidance on the design of illumination.

24.3.2 General Considerations

Illumination is a roadside obstacle that requires appropriate safety treatment. All illumination features, such as poles and foundations, must be checked against the appropriate clear zone requirements. If the illumination feature is deemed crashworthy by the Department, it can be placed within the clear zone without additional roadside safety hardware for protection. Safety treatment may include barrier, metal beam guard fence, cable barrier, or other crashworthy roadside safety hardware. While locating non-breakaway illumination poles outside of the clear zone meets minimum requirements, where feasible, locations that are as near to the ROW as possible are desirable; for example, beyond the front-slope and outside the run-out area of relatively steep slopes. Refer to the applicable **TxDOT Traffic Standards - Illumination** for specific application of illumination features.

Adequate access must be provided within the State ROW for department-maintained illumination systems. Elements such as luminaires, poles, ground boxes, and electric service must be accessible by maintenance vehicles and maintenance personnel. Where practical, it is desirable for the illumination system to be accessed without disrupting existing traffic.

Illumination pole locations must be reviewed to ensure they do not adversely impact hydraulic operation of roadside ditches or roadside maintenance operations.

Existing utilities and subsurface features such as culverts, storm sewer, waterlines, and gas lines can also influence the location of illumination poles. Foundations must be verified with subsurface data to ensure clearance requirements are met.

Lighting systems near special facilities will have specific horizontal and vertical clearance requirements. Special facilities may include railroad tracks, airports, helicopter landing pads, and overhead electric lines.

Illumination should complement the context classification of the facility. The presence of off-site glare or skyglow (i.e, light pollution) near residential areas or nature preserves should be taken into consideration. A photometric analysis should be performed to evaluate these impacts and address community concerns. Local ordinances and aesthetic requirements may dictate the type and design of lighting to be installed, such as decorative poles or luminaires.

24.3.3 Other Safety Benefits

Being able to adequately see the road ahead to observe conflicting traffic and the behavior of other highway users, including where bicycles and pedestrians are present, is integral to driving. Illumination significantly improves the visibility of the roadway, increases sight distance, and makes roadside obstacles more noticeable and therefore more avoidable.

In addition to traffic safety, adequate lighting provides benefits in terms of personal security. Roadway illumination often serves the purpose of safeguarding personal safety for pedestrians, bicyclists, and transit users. Darkness can reduce personal security. Walking, bicycling, or commercial activities may become uncomfortable or unsafe if adequate illumination is not provided. Therefore, ensuring that the lighting provides acceptable levels of illumination can be a great benefit to all users of the roadway environment.

24.4 Signing

24.4.1 Overview

Signing is necessary to direct and protect motorists and pedestrians. Signing provides regulatory conditions and information to alert motorists of risks and direct traffic movement.

Refer to the **TMUTCD**, the **Standard Highway Sign Designs for Texas (SHSD)**, TxDOT's **Sign Crew Field Book**, TxDOT's **Freeway Signing Handbook**, TxDOT's **Sign Guidelines and Applications Manual**, and TxDOT standards for additional information and guidance on the design of signing.

24.4.2 General Considerations

The location of the signs must be visible and unobstructed to motorists and pedestrians for them to effectively communicate information. Horizontal and vertical sight distances must be checked to ensure that the sign is unobstructed and that motorists and pedestrians have adequate time to react to what the sign is communicating.

Large sign structures are a roadside obstacle that require appropriate safety treatment. Large sign structures deemed crashworthy by the Department may be placed within the clear zone without additional safety treatment; otherwise, the appropriate clear zone requirements must be checked to ensure that the proper safety treatment is used.

Large sign structure locations should be reviewed to ensure they do not adversely impact hydraulic operation of roadside ditches. If this cannot be avoided proper scour countermeasures shall be designed and installed to protect the ditch and sign foundation integrity.

Existing utilities and subsurface features such as culverts, storm sewer, waterlines, and gas lines can also influence the location of sign structures. Foundations must be verified with subsurface data to ensure clearance requirements are met.

24.4.3 Other Safety Benefits

Enhanced signing may increase safety of highway facilities in targeted areas with a high frequency of crashes or constrained geometry such as horizontal and/or vertical curves with limited sight distance. Where bicycles and pedestrians are present, enhanced signing should be used to alert drivers to pedestrian crossings and on-road bicycle facilities.

24.5 Pavement Markings

24.5.1 Overview

Pavement markings provide guidance for motorists, bicyclists, and pedestrians as they help guide traffic and limit motorist movement on the roadway. The material, color, and layout (continuous, dashed, etc.) all communicate messages to the motorist which guide their movements. In some cases, this can also be accomplished with words and/or symbols on the roadway.

Refer to the **TMUTCD**, TxDOT's **Pavement Marking Handbook**, and TxDOT traffic standards for specific design requirements for pavement markings.

24.5.2 General Considerations

Lane and shoulder width affect the safety of drivers and level of service of the roadway. **Chapters 5 through 9** of this manual identify the appropriate lane and shoulder widths for various classes of highway facilities. Widths are considered to the center of the striping such that striping widths are excluded from width determinations.

24.5.3 Other Safety Benefits

Enhanced pavement markings may increase safety of highway facilities in targeted areas with a high frequency of crashes or constrained geometry such as horizontal and/or vertical curves with limited sight distance. Where bicycles and pedestrians are present, enhanced pavement markings should be used to alert drivers to pedestrian crossings and on-road bicycle facilities.

24.6 Intelligent Transportation Systems (ITS)

24.6.1 Overview

ITS collect and relay real-time traffic information to improve safety, mobility, security, and integration of transportation systems. This information provides motorists with important and current information as traffic conditions change. The need for ITS devices may vary based on the context classification of the facility.

Refer to TxDOT standards and TxDOT's **Smart Work Zone Guidelines** for additional information and guidance on the design of ITS systems. For electrical design and components, refer to the **National Electric Code**.

24.6.2 General Considerations

Like Illumination, ITS structures are a roadside obstacle and must be checked against the appropriate clear zone requirements to ensure that the proper safety treatment is provided when needed.

Many ITS devices require specific placement and configuration requirements to perform properly. However, accessibility to these devices must be factored in when determining the location. Adequate access must be provided within the State ROW for department-maintained ITS systems. Elements such as cameras, data equipment, cabinets, dynamic message signs, and ground boxes must be accessible by maintenance vehicles and maintenance personnel. Where practical, it is desirable for the ITS system to be accessed without disrupting existing traffic.

Sight distances must be checked for dynamic message signs to ensure that the structure is visible to motorists for the length of time necessary to properly comprehend the more complex messages and that they have adequate time to react to what the sign is communicating.

24.6.3 Other Safety Benefits

Enhanced ITS devices may increase the safety of highway facilities. The addition of dynamic message signs to high-speed roadways may increase safety by alerting drivers to unexpected road conditions. Unexpected road conditions may include construction activities, lane closures, crashes, hazardous road conditions, severe weather alerts, and changes in traffic patterns along evacuation routes (i.e., contraflow).

Enhanced ITS devices should also be considered on non-freeway facilities in urban or urban core contexts with a high frequency of special events. Special event management may be necessary for stadiums, convention centers, concert halls, racetracks, marathons, and bicycle races. Special events may draw unusually high volumes of vehicular, bicycle, and pedestrian traffic in a short period of time. Dynamic message signs may be used to facilitate frequent short-term changes in traffic patterns (i.e., contraflow) and allow for the safe and efficient ingress and egress of traffic.

24.7 Wildlife Crossings

24.7.1 Overview

This section discusses wildlife crossings and includes the following subsections:

- Background;
- Overpass Crossing;
- Underpass Crossing;
- Culvert Crossing;
- Barriers;
- Wildlife Guards; and
- Fencing and Signing.

24.7.2 Background

Wildlife crossings are used to mitigate environmental impacts of a project by providing animals a way to safely cross roadways. These crossings also improve safety by reducing the chances of a collision between animals and drivers. Wildlife crossing structures should be considered when crash data indicates that the benefit of the crossing outweighs the construction and maintenance cost.

The type of wildlife crossing used must be decided based on site-specific criteria. In some cases, an overpass may be necessary to provide large animals safe passage over a roadway. In other cases, a small culvert could be utilized to carry water flow as well as give small animals a crossing underneath the roadway.

Two major factors in selecting the type of wildlife crossing are topographical constraints and minimizing impacts to the existing habitat. Understanding these variables will produce the most effective and environmentally conscious crossing.

Refer to the **FHWA Wildlife Crossing Structures Handbook Design and Evaluation in North America** and TxDOT's **ROW Acquisition Manual** – Chapter 9: Passes, Stock Passes and Cattle Guards for additional information and guidance.

24.7.3 Overpass Crossings

Overpass bridges span over the roadway for both small and large animals, offering free movement across roadways as shown in **Figure 24-1**. Bridge wildlife crossings should be sized based on the types of species needing passage. The designs must consider the surrounding land and features.

Figure 24-1: Wildlife Overpass Crossing
Source: Texas Monthly



24.7.4 Underpass Crossings

Underpass crossings allow animals to pass underneath the roadway without interacting with traffic as shown in **Figure 24-2**. The vertical clearance will vary based on site-specific needs and species. Underpass crossings can also be designed as a dual-purpose water crossing.

Figure 24-2: Wildlife Underpass Crossing



24.7.5 Culvert Crossings

Culvert wildlife crossings allow passage for wildlife to cross under roadways, railways, and embankments and can improve safety for the motoring public. These crossings utilize pipe or box culverts and are similar to underpass crossings, allowing movement beneath the roadway. These crossings can be designed in areas without water flow or at existing culvert crossings with water flow.

These types of wildlife crossing structures can be sized and located through coordination by the roadway designers with the District Environmental Coordinator and the ENV Division. Details can be obtained from the ENV Natural Resource Management Section and District Environmental Coordinators. When preparing structure plan sheets for cast-in-place or precast box culverts, designers should note that these culverts can have single or multiple openings allowing the passage of water, livestock, or wildlife under a roadway.

24.7.5.1 Culverts without Water Flow

These crossings make use of culvert structures to provide wildlife passage underneath a roadway. Box culverts or pipes can be used depending on the specific needs of the project as shown in **Figure 24-3**. Sizing of the crossing should be based on the wildlife intended to use the crossing. Refer to **TxDOT Wildlife Accommodation Standards** for additional information.

Figure 24-3: Culvert Wildlife Crossing Without Water Flow



24.7.5.2 Culverts with Water Flow

These crossings incorporate box culverts and pipes to allow both wildlife and water to cross underneath the roadway as shown in **Figure 24-4**. The crossing should be sized based on hydraulic data and capacity requirements as well as on the size of the animals which will use the crossing. These crossings may also include dry walkways (1-ft to 2-ft high and 1.5-ft to 2-ft wide) within the culvert to allow wildlife easier

passage through the structure. Refer to **TxDOT Bridge Standards - Wildlife Accommodations** for additional information.

Figure 24-4: Culvert Wildlife Crossing with Water Flow and Dry Walkways



In some cases, culvert crossings may need to accommodate fish or other aquatic organisms. To facilitate passage for these types of organisms, lower hydraulic velocities and culvert slopes should be considered.

24.7.6 Barriers

When barrier or rail is used, it may present a challenge for wildlife to traverse when crossing a highway facility. In these areas, the designer may consider using Hydraulically Efficient Barrier (HEB). While this type of barrier is generally used to increase hydraulic efficiency, it may also be used to allow smaller wildlife to cross the highway more easily due to the larger slot openings at the bottom of the barrier. Refer to **TxDOT Roadway Standards** for additional information and design details on HEBs.

24.7.7 Wildlife Guards

Wildlife guards, with or without gates, may be used to:

- Prevent livestock from entering the ROW and interfering with roadway traffic;
- Maintain range control; and
- Reduce animal-vehicle conflicts for wildlife.

To prevent livestock and wildlife from entering the ROW, construction of wildlife guards, often accompanied by fencing at side roads and private entrances, can be a cost-effective way to improve safety on a roadway. When placed near traffic interchanges on a crossroad, wildlife guards without gates should be placed at or near the access control line to prevent livestock and wildlife from entering a main roadway. The number of units will be determined by the width of the roadway, the number of private drives that need to access State ROW, and the types of wildlife or livestock that may enter the ROW. Some Districts have developed new types of wildlife guards with different types of bars and checked grates to reduce wildlife egress from private driveways. Designers can work with their District Environmental Coordinators and the ENV Division to determine the appropriate types of wildlife guards and fencing. See **Figure 24-5** for an example of a grated wildlife guard and **Figure 24-6** for an example of a round bar wildlife guard.

Figure 24-5: District Wildlife Guard Example 1



Figure 24-6: District Wildlife Guard Example 2



Source: FHWA Wildlife Crossing Structure Handbook Design and Evaluation in North America

24.7.8 Fencing and Signing

Fencing should be used to guide wildlife toward the crossing structure. The new structure will be unfamiliar to wildlife in the area and they may stray from using it and cross the roadway instead. The addition of fencing should help guide the wildlife along the crossing until they become comfortable using the crossing regularly. If the fencing is temporary the scheduled removal date should be specified in the plans.

Routine maintenance ensures the effectiveness of wildlife crossings. If left alone, the crossings could become overgrown or unusable. Coordination is required with Texas Parks and Wildlife Department for specific design requirements. Design crossings and culverts are required to meet TxDOT standards. For a further breakdown of different crossing designs, refer to the **FHWA Wildlife Crossing Structures Handbook Design and Evaluation in North America**.

Signing is a cost-effective safety countermeasure that can be used to alert motorists of potential presence of wildlife on or near the highway. The goal of these types of signs is to reduce the likelihood of vehicle collisions with wildlife and encourage drivers to be more alert.

See **Section 24.3** for additional information and guidance on the design of signing.

24.8 Texas Aggregate Quarry and Pit Safety Act

The purpose of the Texas Aggregate Quarry and Pit Safety Act is to protect the travelling public by requiring safety devices for certain aggregate quarries and pits. The following items would not be subject to the provisions of this Act: A quarry or pit dug solely for the purpose of constructing a stock pond unless the pond was originally dug as a quarry or pit; a detention pond constructed for a municipality, or a municipal drainage-way. For additional information, reference **Texas Aggregate Quarry and Pit Safety Act** Chapter 133 – Natural Resources Code, and **43 TAC, Part 1, Chapter 21, Subchapter M**

Appendix A
Minimum Design Criteria for Projects Requiring a Design Exception

Table A-1: 4R Design Speed (Minimum)

Table A-2: 4R Lane Width (Minimum)

Table A-3: 4R Shoulder Width (Minimum)[1,2,3,4]

Table A-4: 4R Horizontal Curve Radius (Minimum)

Table A-5: 4R Stopping Sight Distance (SSD) (Minimum)

Table A-6: 4R Maximum Grade

Table A-7: 4R Cross Slope (Maximum)

Table A-8: 4R Vertical Clearance (Minimum)[1]

Table A-9: 4R Design Loading Structural Capacity (Maximum Loadings)

Table A-10: 4R Bridge Class Culvert Protection

Table A-11: 4R Bridge Rail

Table A-12: 3R Design Speed (Minimum)[1]

Table A-13: 3R Lane Width (Minimum)[1]

Table A-14: 3R Shoulder Width (Minimum)[1]

Table A-15: 3R Horizontal Curve Radius (Minimum)

Table A-16: 3R Stopping Sight Distance (SSD) (Minimum)[1]

Table A-17: 3R Maximum Grades

Table A-18: 3R Design Loading Structural Capacity (Maximum Loadings)[1]

Table A-19: 3R Bridge Class Culvert Protection[1]

Table A-20: 3R Deficient Bridge Rail[1]

Table A-21: Bike Lane Width (Minimum)

Table A-22: Bike Shared Lane Width (Minimum)

Table A-23: Bridge Deck Clear Space (Minimum)[1]

Table A-24: TxDOT's Controlling Criteria for Off-System Bridges with ≤ 400 ADT

Table A-25: TxDOT's Controlling Criteria for Off-System Historically Significant Bridges

Table A-1: 4R Design Speed (Minimum)

Facility Type	Additional Factors	Design Speed	Reference
Freeways	Rural	70 MPH	TxDOT Criteria
	Suburban	60 MPH	TxDOT Criteria
	Urban	50 MPH	TxDOT Criteria
	Urban Core	50 MPH	AASHTO pg. 8-2[4]
Arterials	Rural (Level Terrain)	70 MPH	TxDOT Criteria
	Rural (Rolling Terrain)	60 MPH	
	Rural Town	30 MPH	
	Suburban	40 MPH	
	Urban	30 MPH	
	Urban Core	25 MPH	
Collector	Rural (Level Terrain)		TxDOT Criteria
	ADT < 400	50 MPH[1]	
	ADT 400 - 2000	50 MPH	
	ADT > 2000	60 MPH	
	Rural (Rolling Terrain)		
	ADT < 400	40 MPH[2]	
	ADT 400 - 2000	40 MPH	
	ADT > 2000	50 MPH	
	Rural Town	30 MPH	TxDOT Criteria
	Suburban	35 MPH	AASHTO pg. 6-13[4]
	Urban	30 MPH	
	Urban Core	25 MPH	
Local Roadways[3]	Rural (Level Terrain)		TxDOT Criteria
	ADT < 400	40 MPH[2]	
	ADT 400 - 2000	50 MPH	
	ADT > 2000	50 MPH	
	Rural (Rolling Terrain)		
	ADT < 400	30 MPH	
	ADT 400 - 2000	40 MPH	
	ADT > 2000	40 MPH	
	Rural Town	25 MPH	TxDOT Criteria
	Suburban	30 MPH	
	Urban	25 MPH	
	Urban Core	20 MPH	

Facility Type	Additional Factors	Design Speed	Reference
Ramps and Direct Connectors	Highway Design Speed: 30 MPH 35 MPH 40 MPH 45 MPH 50 MPH 55 MPH 60 MPH 65 MPH 70 MPH 75 MPH 80 MPH	 15 MPH 20 MPH 20 MPH 25 MPH 25 MPH 30 MPH 30 MPH 30 MPH 35 MPH 40 MPH 40 MPH	TxDOT Criteria
Collector-Distributor Roads	Rural Suburban Urban	60 MPH 50 MPH 40 MPH	AASHTO pg. 8-39[4]
Frontage Roads	Use Collector Criteria		TxDOT Criteria

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. Referenced from AASHTO A Policy on Geometric Design of Highways and Streets, 2018, 7th Edition.

Table A-2: 4R Lane Width (Minimum)

Facility Type	Additional Factors	Lane Width	Reference
Freeways	Rural Suburban Urban Urban Core	12-ft	TxDOT Criteria
Arterials	Rural:[5,6,7,8] 2 lane: ADT < 400 ADT 400 - 2000 ADT > 2000	12-ft	TxDOT Criteria
	Rural:[5,6,7,8] 4 lane divided 6 lane divided 4 lane undivided	12-ft	
	Rural Town[1] Suburban[1] Urban[1] Urban Core[1]	11-ft	
Collector	Rural[5,6,7,8]: 2 lane:		TxDOT Criteria
	ADT < 400 (DS ≤ 55 MPH)[2] ADT < 400 (DS > 55 MPH)[2]	10-ft 11-ft	
	ADT 400 - 1500 (DS ≤ 70 MPH)[2] ADT 400-1500 (DS > 70 MPH)[2]	11-ft 12-ft	
	ADT 1500 - 2000 (DS ≤ 45 MPH)[2] ADT 1500 - 2000 (DS > 45 MPH)[2] ADT > 2000	11-ft 12-ft 12-ft	
	Rural:[5,6,7,8] 4 lane divided 6 lane divided 4 lane undivided	12-ft	
	Rural Town[1] Suburban[1] Urban[1] Urban Core[1]	11-ft	

Facility Type	Additional Factors	Lane Width	Reference
Local[4]	Rural[5,6,7,8]: ADT < 400 ADT 400 - 1500 ADT 1500 - 2000 ADT > 2000	 10-ft 11-ft 11-ft 12-ft	TxDOT Criteria
	Rural[5,6,7,8]: 4 lane divided 6 lane divided 4 lane undivided	12-ft	
	Rural Town[3] Suburban[3] Urban[3] Urban Core[3]	11-ft	
Ramps (uncurbed) and Direct Connectors	Rural Suburban Urban	14-ft[9]	TxDOT Criteria
Ramps (curbed)	Rural Suburban Urban	22-ft[9]	TxDOT Criteria
Collector-Distributor Roads	Use Ramp Criteria		
Frontage Roads	Use Collector Criteria		

Facility Type	Additional Factors	Lane Width	Reference
Notes:			
1. In highly restricted locations or locations with less than 5% trucks and speeds less than or equal to 40 mph, 10-ft lane is permissible. Engineering judgement should be exercised when determining if 10-ft is acceptable and must be approved by the District. Use of 10-ft travel lanes must be approved by the District and approval documented in the project file. If the District approves the use of 10-ft lanes, a design exception would not be required.			
2. "DS" stands for Design Speed			
3. 9-ft minimum lane width in residential areas. 10-ft minimum lane width in non-industrial areas. 11-ft minimum lane width in industrial areas.			
4. Applicable only to off-system routes that are not functionally classified at a higher classification.			
5. 24-ft minimum surfacing width for all on-system state highway routes.			
6. 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 ≥ 2000 ADT in reservoir areas, two 12-ft lanes with 10-ft shoulders should be used.			
7. Minimum width of new or widened structures should accommodate the approach roadway including shoulders.			
8. A minimum surfacing width of 36-ft should be considered on horizontal curves with a radius less than 190-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.			
9. 12-ft lanes are permissible on ramps and direct connectors with multiple lanes.			

Table A-3: 4R Shoulder Width (Minimum)[1,2,3,4]

Facility Type	Additional Factors	Inside Shoulder	Outside Shoulder	Reference
Freeways	4 lanes	4-ft paved	10-ft paved	TxDOT Criteria
	6 lanes or more	10-ft paved	10-ft paved	
Arterials[5,10,15]	Rural:			TxDOT criteria
	2 lane:			
	ADT < 400	---	4-ft[9]	
	ADT 400 - 1500		6-ft	
	ADT 1500 - 2000		8-ft	
	ADT > 2000		10-ft[11]	
	Rural:			
	4 lane divided	4-ft paved	8-ft	
	6 lane divided	4-ft paved[17]	8-ft	
	4 lane undivided	---	8-ft	
	Rural Town	---	4-ft[9]	
	Suburban		4-ft[9]	
	Urban		4-ft[9]	
	Urban Core		4-ft[9]	
Collector[7,10]	Rural:			TxDOT criteria
	2 lane:			
	ADT < 400 (DS ≤ 45 MPH)[2]	---	2-ft[6,9]	
	ADT < 400 (DS > 45 MPH)[2]		2-ft[6,9]	
	ADT 400 - 1500 (DS ≤ 45 MPH)[2]		4-ft[9]	
	ADT 400 - 1500 (DS > 45 MPH)[2]		6-ft	
	ADT 1500 - 2000 (DS ≤ 45 MPH)[2]		8-ft	
	ADT 1500 - 2000 (DS > 45 MPH)[2]		8-ft	
	ADT > 2000		10-ft[11]	
	Rural:			
	4 lane divided	4-ft paved	8-ft	
	6 lane divided	4-ft paved[17]	8-ft	
	4 lane undivided	---	8-ft[14]	
	Rural Town	---	3-ft[6,9]	
	Suburban		4-ft[9]	
	Urban		3-ft[6,9]	
	Urban Core		3-ft[6,9]	

Facility Type	Additional Factors	Inside Shoulder	Outside Shoulder	Reference
Local[8,10]	Rural: 2 lane: ADT < 400 ADT 400 - 2000 ADT > 2000	---	 2-ft[6,9] 4-ft[9,12] 8-ft	TxDOT criteria
	Rural Town Suburban Urban Urban Core	--- 	2-ft[6,9] 4-ft[9] 2-ft[6,9] 2-ft[6,9]	TxDOT criteria
Ramps and Direct Connectors[13]	Rural Suburban Urban	2-ft or 4-ft[6] Rdwy; 4-ft[6]Str	6-ft[16]	TxDOT Criteria
Collector-Distributor Roads	Use Mainlane Criteria			AASHTO pg. 10-95[18]
Frontage Roads	Rural Two-Way: < 400 ADT 400 - 1500 ADT 1500 – 2000 ADT > 2000 ADT	 2-ft 4-ft 8-ft 8-ft		TxDOT Criteria
	Rural One-Way: < 400 ADT 400 – 1500 ADT 1500 – 2000 ADT > 2000 ADT	 2-ft 2-ft 4-ft 4-ft	 2-ft 4-ft 8-ft 8-ft	TxDOT Criteria
	Rural Town Suburban Urban Urban Core	Use Collector Criteria		TxDOT Criteria

Facility Type	Additional Factors	Inside Shoulder	Outside Shoulder	Reference
Notes:				
1. Full minimum width of shoulder should be fully surfaced. Where wider shoulder widths are used, partial (not less than minimum width) surfacing or full width surfacing may be provided at the option of the designer.				
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 ≥ 2000 ADT in reservoir areas, two 12-ft lanes with 10-ft shoulders should be used.				
3. Minimum width of new or widened structures should accommodate the approach roadway including shoulders.				
4. 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.				
5. On arterials, shoulders should be fully surfaced.				
6. 4-ft minimum shoulder if longitudinal barrier is used. 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.				
7. On collectors, shoulders must be fully surfaced on collectors with ≥ 1500 ADT. Shoulder surfacing not required but is desirable even if only for minimum width on collectors with < 1500 ADT on all local roads.				
8. Applicable only to off-system routes that are not functionally classified at a higher classification.				
9. 5-ft minimum clear space (shoulder) for bicyclists should be provided on bridges being replaced or rehabilitated, except on off-system facilities with less than 400 ADT. For off-system bridges with current ADT greater than 400 ADT, the 5-ft additional shoulder width may be excepted through the Design Exception process from the bicycle clear space requirement if the additional width represents an unreasonable increase in cost.				
10. Where left or right turn lanes are present on uncurbed facilities, a 4-ft fully surfaced shoulder must be provided.				
11. In highly constrained conditions 8-ft may be used with engineering judgement, 10-ft is ideal.				
12. Mailbox Turnouts should be considered on local roads with ≥ 400 ADT. Refer to TxDOT standard drawings for details.				
13. 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 6-ft minimum 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 6).				
14. On 4-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.				
15. Arterials on the Texas Highway Freight Network (THFN) should provide 10-ft minimum outside shoulders.				
16. For Direct Connectors 8' minimum outside shoulder is required. No minimum shoulder with required on curbed ramps.				
17. On 6 lane or more divided arterials, 10-ft inside shoulders should be provided. The inside shoulder width may be reduced to 4-ft in constrained situations.				
18. Referenced from AASHTO A Policy on Geometric Design of Highways and Streets, 2018, 7th Edition.				

Table A-4: 4R Horizontal Curve Radius (Minimum)

Facility Type	Superelevation, e-max	Curve Radius (feet) for Design Speed (mph)															Reference
		15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Low-speed Rural Town, Suburban, urban and Urban Core Facilities	0.04	42	86	154	250	371	533	711	---	---	---	---	---	---	---	---	TxDOT Criteria
	NC	50	107	198	333	510	762	1039									Method 2[1]
High-speed Suburban and Urban Non-Freeway Facilities	0.04	---	---	---	---	---	---	---	926	1190	1500	---	---	---	---	---	TxDOT Criteria
Rural and High-speed Suburban, urban and Urban Core Facilities	0.06	39	81	144	231	340	485	643	833	1060	1330	1660	2040	2500	3050	---	AASHTO Tables 3-9, & 3-10[3]
	0.08	38	76	134	214	314	444	587	758	960	1200	1480	1810	2210	2670	---	Method 5[2]

Notes:

1. AASHTO uses Method 2 superelevation distribution. Method 2 only introduces superelevation after the maximum side friction has been used. Therefore, no superelevation is needed on flatter curves. This method is used on low-speed rural town, suburban, urban and urban core facilities where, because of various constraints, superelevation frequently cannot be provided.

2.Method 5 is intended to accommodate overdriving that is likely to occur on flat to intermediate curves. Overdriving on such curves involves little risk that a driver will lose control of the vehicle because superelevation sustains nearly all the lateral acceleration at the average running speed and a large amount of side friction is available for greater speeds.

3. Referenced from AASHTO A Policy on Geometric Design of Highways and Streets, 2018, 7th Edition.

Table A-5: 4R Stopping Sight Distance (SSD) (Minimum)

	Design Speed (mph)															Reference
	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Stopping Sight Distance (feet) for Design[1]	80	115	155	200	250	305	360	425	495	570	645	730	820	910	1010	AASHTO pg. 3-4, Table 3-1[2]

Notes:

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. 4-6), the vertical SSD is satisfied under the usual conditions.

2. Referenced from AASHTO A Policy on Geometric Design of Highways and Streets, 2018, 7th Edition.

Table A-6: 4R Maximum Grade

Facility Type	Additional Factors	Terrain	Maximum Grade (%) for Design Speed (mph)															Reference
			15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Freeway	Rural and Urban	Level	---	---	---	---	---	---	---	4	4	3	3	3	3	3	---	TxDOT Criteria
		Rolling	---	---	---	---	---	---	---	5	5	4	4	4	4	4	---	
Arterial	Rural	Level	---	---	---	---	---	5	5	4	4	3	3	3	3	3	---	TxDOT Criteria
		Rolling	---	---	---	---	---	6	6	5	5	4	4	4	4	4	---	
	Urban	Level	---	---	---	8	7	7	6	6	5	5	---	---	---	---	---	TxDOT Criteria
		Rolling	---	---	---	9	8	8	7	7	6	6	---	---	---	---	---	
Collector	Rural	Level	---	7	7	7	7	7	7	6	6	5	---	---	---	---	---	TxDOT Criteria
		Rolling	---	10	10	9	9	8	8	7	7	6	---	---	---	---	---	
	Urban	Level	9	9	9	9	9	9	8	7	7	6	---	---	---	---	---	TxDOT Criteria
		Rolling	12	12	12	11	10	10	9	8	8	7	---	---	---	---	---	
Local	Rural	Level	9	8	7	7	7	7	7	6	6	5	---	---	---	---	---	TxDOT Criteria
		Rolling	12	11	11	10	10	10	9	8	7	6	---	---	---	---	---	
	Rural Town, Suburban, Urban, Urban Core[1]	All	8	8	8	8	8	8	8	---	---	---	---	---	---	---	---	TxDOT Criteria
													---	---	---	---	---	

Notes:

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

Table A-7: 4R Cross Slope (Maximum)

Facility Type	Additional Factors	Cross Slope (ft/ft)	Reference
Freeways	All Criteria	0.02[2]	AAHSTO pg. 8-3[3]
Arterials	Rural	0.02	AAHSTO pg. 7-6[3]
	Rural Town	0.03	AASHTO pg. 7-39[3]
	Suburban	0.03	
	Urban	0.03	
	Urban Core	0.03	
	Divided Arterials	0.02[1]	AASHTO pg. 7-15, 16[3]
Collector	Rural	0.02	AASHTO pg. 6-4[3]
	Rural Town	0.03	AASHTO pg. 6-15[3]
	Suburban	0.03	
	Urban	0.03	
	Urban Core	0.03	
Local	Paved	0.02	AASHTO pg. 5-4[3]
	Unpaved	0.06	
Shoulders	Paved	0.06	AASHTO pg. 4-13[3]
	Gravel	0.06	
	Turf	0.08	
Ramps		0.02	AASHTO pg. 10-111[3]
Frontage Roads	Use Collector Criteria		

Notes:

1. Values given is for up to two lanes in one direction. Additional outside lanes may have cross slopes up to 0.03.
2. For areas of heavy rainfall, a pavement cross slope of .025 may be used to provide adequate drainage.
3. Referenced from AASHTO A Policy on Geometric Design of Highways and Streets, 2018, 7th Edition.

Table A-8: 4R Vertical Clearance (Minimum)[1]

Facility Type	Vehicle Overpasses[2]	Overhead Sign Structures and Bicycle/Pedestrian Overpasses[4]	Reference
Freeway	16.5-ft	17.5-ft	TxDOT Criteria
Arterial Collector Local	16.5-ft[3]	17.5-ft	TxDOT Criteria
Texas Highway Freight Network (THFN)[5]	18.5-ft	19.5-ft	TxDOT Criteria
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. Includes overhead signs, light fixtures, sign bridges, and toll gantries.			
5. If an overlay reduces the vertical clearance below 18-ft, a TFHN Design Deviation will be required.			

Table A-9: 4R Design Loading Structural Capacity (Maximum Loadings)

Facility Type	Reference
Freeway Arterial Collector Local	See TxDOT Bridge Division Manual - LRFD for maximum loadings.

Table A-10: 4R Bridge Class Culvert Protection

Facility Type	Condition	Reference
Freeway Arterial Collector Local	All ADT's	TxDOT Bridge Railing Manual

Table A-11: 4R Bridge Rail

Facility Type	Condition	Reference
Freeway Arterial Collector Local	Bridge rail must comply with MASH and meet requirements of TxDOT Bridge Railing Manual	TxDOT Bridge Railing Manual

Table A-12: 3R Design Speed (Minimum)[1]

Facility Type	Additional Factors	Design Speed	Reference
Freeways	---- ----	---- ----	See Table A-1 for 4R Criteria Values
Arterials & Collectors	Rural: 2-Lane	40 MPH	TxDOT Criteria
	4-Lane Undivided 4-Lane Divided 6-Lane Divided	50 MPH	
	Rural Town Suburban Urban Core Urban	30 MPH	
Frontage Roads	Rural	40 MPH	TxDOT Criteria
	Suburban Urban	30 MPH	
Notes:			
1. Applicable to high volume (ADT ≥ 1500) roadways.			

Table A-13: 3R Lane Width (Minimum)[1]

Facility Type	Additional Factors	Lane Width	Reference
Freeways	---- ---- ----	----	See 4R Criteria Values
Arterials and Collectors	Rural: 2-Lane ADT < 400 ADT ≥ 400	 10-ft 11-ft	TxDOT Criteria
	4-Lane Undivided 4-Lane Divided 6-Lane Divided	11-ft	
	Rural Town Suburban Urban Core Urban	10-ft	TxDOT Criteria
Frontage Roads	Rural: ADT < 1500 ADT ≥ 1500	 10-ft 11-ft	TxDOT Criteria
	Suburban Urban	10-ft	
Notes:			
1. Applicable to low volume and high volume roadways.			

Table A-14: 3R Shoulder Width (Minimum)[1]

Facility Type	Additional Factors	Inside Shoulder	Outside Shoulder	Reference
Freeways	----------	----------	----------	See Table A-3 for 4R criteria values
Arterials & Collectors	Rural: 2 lane: ADT < 400 ADT 400 - 1500 ADT ≥ 1500 4 lane divided 6 lane divided 4 lane undivided	 --- --- --- 2-ft 4-ft ---	 0-ft 1-ft 3-ft 4-ft 4-ft 4-ft	TxDOT criteria
	Rural Town Suburban Urban Urban Core	---	2-ft[2]	TxDOT criteria
Frontage Roads	Rural: Current ADT < 1500 2-way operation 1-way operation Current ADT ≥ 1500 2-way operation 1-way operation	 1-ft 2-ft	 1-ft 1-ft 3-ft 4-ft	TxDOT Criteria
	Suburban Urban	---	2-ft	TxDOT Criteria

Notes:

1. Applicable to low volume and high volume roadways.
2. Shoulders width applicable to uncurbed sections, 0-ft minimum for curbed sections.

Table A-15: 3R Horizontal Curve Radius (Minimum)[1]

Facility Type	Superelevation, e-max	Curve Radius (feet) for Design Speed (mph)														Reference
		15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Low-speed Rural Town, Suburban, urban and Urban Core Facilities	0.04	--	--	--	--	--	--	--	---	---	---	---	---	---	---	---	See Table A-4 for 4R Criteria
	NC	--	--	--	--	--	--	--									
High-speed Suburban and Urban Facilities	0.04	---	---	---	---	---	---	---	--	--	--	---	---	---	---	---	See Table A-4 for 4R Criteria
Rural and High-speed Suburban, urban and Urban Core Facilities	0.06	--	--	--	--	--	--	--	--	--	--	---	--	--	--	---	See Table A-4 for 4R Criteria
	0.08	--	--	--	--	--	--	--	--	--	--	--	--	--	--	---	

Notes:

1. Applicable to high volume (ADT ≥ 1500) roadways.

Table A-16: 3R Stopping Sight Distance (SSD) (Minimum)[1]

Facility Type	Design Speed (mph)															Reference
	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Stopping Sight Distance (feet) for Design	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	See Table A-6 for 4R Criteria
Notes:																
1. Applicable to high volume (ADT ≥ 1500) roadways.																

Table A-17: 3R Maximum Grades[1]

Facility Type	Additional Factors	Terrain	Maximum Grade (%) for Design Speed (mph)															Reference
			15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Freeway	Rural and Urban	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
Arterial	Rural	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
	Urban	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
Collector	Rural	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
	Urban	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
Local	Rural	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
	Rural Town, Suburban, Urban, Urban Core	All	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria
													---	---	---	---	---	

Notes:

1. Applicable to high volume (ADT ≥ 1500) roadways

Table A-18: 3R Design Loading Structural Capacity (Maximum Loadings)[1]

Facility Type	Reference
Freeway Arterial Collector Local	See TxDOT Bridge Design Manual - LRFD for minimum loadings.
Notes:	
1. Applicable to low volume and high volume roadways.	

Table A-19: 3R Bridge Class Culvert Protection[1]

Facility Type	Condition	Reference
Freeway Arterial Collector Local	All ADT's	TxDOT Bridge Railing Manual
Notes:		
1. Applicable to high volume (ADT ≥ 1500) roadways		

Table A-20: 3R Deficient Bridge Rail[1]

Facility Type	Condition	Reference
Freeway Arterial Collector Local	Minimum requirements are met for new bridge rail or existing rail to remain in place.	TxDOT Bridge Railing Manual
Notes:		
1. Applicable to high volume (ADT ≥ 1500) roadways		

Table A-21: Bike Lane Width (Minimum)

Facility Type	Additional Factors	Width	Reference
Arterials Collectors Local Frontage Roads	Rural	----	See Ch. 18, Section 18.4.4.2
	Rural Town, Suburban, Urban, Urban Core	----	

Table A-22: Bike Shared Lane Width (Minimum)

Facility Type	Additional Factors	Width	Reference
Arterials Collectors Local Frontage Roads	Rural, Rural Town, Suburban, Urban, Urban Core	13-ft	See Ch. 18, Section 18.4.7.2

Table A-23: Bridge Deck Clear Space (Minimum)[1]

Facility Type	Additional Factors	Width	Reference
Arterials Collectors Local Frontage Roads	All Contexts	5-ft[2,3]	TxDOT Criteria
Notes:			
1. Off-system bridges, with current ADT greater than 400 ADT, where this may represent an unreasonable increase in cost may be exempt from this requirement, See Chapter 18 for specific guidance . See Chapter 12.2 for specific off-system bridge requirements for current ADT of 400 or less.			
2. 4-ft shoulder + 1-ft offset measured to the toe of barrier. A shared use path may be used in lieu of clear space.			
3. 8-ft minimum shoulder width should be provided on bridges for roadways identified on the Texas Bicycle Tourism Trails Example Network.			

Table A-24: TxDOT's Controlling Criteria for Off-System Bridges with ≤ 400 ADT

Facility Type	Criteria	Minimum
Local (Off-system)	Design Speed	Meet or improve conditions that are typical on the remainder of the roadway.
	Stopping Sight Distance (SSD)[1]	Meet or improve conditions that are typical on the remainder of the roadway.
	Horizontal Curve Radius	Meet or improve conditions that are typical on the remainder of the roadway.
	Superelevation Rate	Meet or improve conditions that are typical on the remainder of the roadway.
	Maximum Grade	Meet or improve conditions that are typical on the remainder of the roadway.
	Minimum Structure Width (face to face of rail)	24-ft[2]
		28-ft[3]
	Bridge End Guard Fence	Current applicable Roadway Design Standards.
	Approach Roadway Width	Match bridge roadway width for 50-ft minimum. Minimum 50-ft transition length to country road width. If surfacing is included, 50-ft minimum length adjacent to bridge should be surfaced the width of the bridge roadway width.
	Traffic Control	Detours should match county road design features. Texas MUTCD requirements for traffic control device placement.
	Design Loading Structural Capacity	See TxDOT Bridge Division Manual - LRFD for minimum loadings.

Notes:

1. Stopping Sight Distance (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 K-Value meets or improves the conditions of the remainder of the roadway, then the vertical SSD is satisfied under usual conditions.
2. For approach roadways ≤ 24-ft.
3. For approach roadways > 24-ft, or the county or municipal entity has near term plans to widen the approach roadway or anticipates near term development.

Table A-25: TxDOT's Controlling Criteria for Off-System Historically Significant Bridges

Facility Type	Criteria	Minimum
Local (Off-system)	Roadway Width	Meet or improve conditions that are typical on the remainder of the roadway.
	Load Carrying Capacity (Operating Rating)	See TxDOT Historic Bridge Manual

Appendix B
Minimum Design Criteria for Projects Requiring a Design Waiver

Table B-1: 4R Curb Parking Lane Width (Minimum)

Table B-2: 4R Speed Change Lane Width (Minimum)

Table B-3: 4R Length of Speed Change Lanes (Minimum)

Table B-4: 4R Curb Offset (Minimum)

Table B-5: 4R Bridge Width (Minimum)

Table B-6: 4R Median Width (Minimum)

Table B-7: 4R Clear Zone (Minimum)

Table B-8: 4R Lateral Offset to Obstructions (Minimum)

Table B-9: 4R Railroad Overpass Geometrics

Table B-10: 4R Sag Vertical Curve Length[1]

Table B-11: 4R Superelevation (Non-rate elements)

Table B-12: 4R Guardrail Length[1]

Table B-13: TxDOT's 4R Non-Controlling Criteria for Bicycle Facilities (Minimum)

Table B-14: 3R Design Speed (Minimum)[1]

Table B-15: 3R Horizontal Curve Radius (Minimum)

Table B-16: 3R Stopping Sight Distance (SSD) (Minimum)[1]

Table B-17: 3R Maximum Grade[1]

Table B-18: 3R Superelevation (Non-rate elements)[1]

Table B-19: Bridge Width[1]

Table B-20: Deficient Bridge Rail[1]

Table B-21: 3R Clear Zone (Minimum)[1]

Table B-22: 3R Speed Change Lane Width (Minimum)[1]

Table B-23: 3R Length of Speed Change Lanes (Minimum)[1]

Table B-24: 3R Curbside Parking Lane Width (Minimum)[1]

Table B-25: 3R Guardrail Length[1,2]

Table B-26: 3R Sag Vertical Curve Length[1]

Table B-1: 4R Curb Parking Lane Width (Minimum)

Facility Type	Additional Factors	Lane Width	Reference
Arterials	Rural Town Urban Core Urban	10-ft[1,3]	TxDOT Criteria
Collectors	Rural Town Urban Core Urban	8-ft[1,3]	TxDOT Criteria
Local	Rural Town Urban Core Urban	8-ft[2]	TxDOT Criteria
Frontage Roads	See Section 4.10.16.4		TxDOT Criteria
Notes:			
1. Where there is no demand for a future through lane, 8-ft minimum may be used.			
2. In non-commercial and non-industrial areas, 7-ft minimum may be used.			
3. If bicyclists and parked vehicles need to be accommodated 12 to 14 -ft widths should be used.			

Table B-2: 4R Speed Change Lane Width (Minimum)

Facility Type	Additional Factors	Lane Width	Reference
Freeways	Rural Suburban Urban Core Urban	12-ft[1]	TxDOT Criteria
Arterials	Rural Rural Town Suburban Urban Core Urban	10-ft	TxDOT Criteria
Collectors	Rural Rural Town Suburban Urban Core Urban	10-ft	TxDOT Criteria
Local	Rural Town Suburban Urban Core Urban	9-ft	TxDOT Criteria
Two-way Left Turns	See Table 4-22		TxDOT Criteria
Frontage Roads	Use Collector Data		TxDOT Criteria
Notes:			
1. 12-ft lane + 6-ft minimum adjacent shoulder			

Table B-3: 4R Length of Speed Change Lanes (Minimum)

Facility Type	Additional Factors	Reference
Arterials	Rural Rural Town Suburban Urban Core Urban	TxDOT Criteria See Table 4-14
Collectors	Rural Rural Town Suburban Urban Core Urban	TxDOT Criteria See Table 4-14
Local	Rural Town Suburban Urban Core Urban	TxDOT Criteria See Table 4-14
Frontage Roads	Use Collector Data	TxDOT Criteria

Table B-4: 4R Curb Offset (Minimum)

Facility Type	Additional Factors	Offset	Reference
Arterials Collectors Local	Rural Town Suburban Urban Core Urban	1-ft	TxDOT Criteria
Frontage Roads	Use Collector Data		TxDOT Criteria

Table B-5: 4R Bridge Width (Minimum)

Facility Type	Reference
Freeway Arterial Collector Local	See TxDOT Bridge Project Development Manual

Table B-6: 4R Median Width (Minimum)

Facility Type	Additional Factors	Median Width	Reference
Freeway	----	----	See chapter 8 section 8.1.9 for freeway median guidance
Arterials Collectors Local	Rural: Flush Depressed	 4-ft 48-ft	TxDOT Criteria
	Rural Town: Suburban: Urban Core: Urban: Raised Raised W/ Pedestrian Refuge Flush Two-Way Left Turn Lanes: DS ≤ 40 mph DS = 45 or 50 mph DS > 50 mph	 4-ft 6-ft[1] 4-ft ---- 11-ft 12-ft 14-ft	
Notes:			
1.). 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.			

Table B-7: 4R Clear Zone (Minimum)

Facility Type	Additional Factors	Clear Zone Width	Reference
Freeways	Rural Suburban	30-ft	TxDOT Criteria
Arterials	Rural: ADT < 750 ADT ≥750	 16-ft 30-ft	TxDOT Criteria
	Suburban: ADT ≤ 16000 ADT > 16000	 10-ft 20-ft	
	Urban: Curbed (DS ≥ 50 MPH) Uncurbed (DS ≥ 50 MPH)	 Use Suburban Criteria	
	Rural Town, Urban, Urban Core: Curbed (DS ≤ 45 MPH) Uncurbed (DS ≤ 45 MPH)	 4-ft from FOC 10-ft	
Collector	Rural: DS ≤ 45 MPH DS ≥ 50 MPH (ADT ≤ 750) DS ≥ 50 MPH (ADT > 750)	 10-ft 16-ft 30-ft	TxDOT Criteria
	Suburban: ADT ≤ 16000 ADT > 16000	 10-ft 20-ft	
	Urban: Curbed (DS ≥ 50 MPH) Uncurbed (DS ≥ 50 MPH)	 Use Suburban Criteria	
	Rural Town, Urban, Urban Core: Curbed (DS ≤ 45 MPH) Uncurbed (DS ≤ 45 MPH)	 4-ft from FOC 10-ft	

Facility Type	Additional Factors	Clear Zone Width	Reference
Local Roadways	Rural	10-ft	TxDOT Criteria
	Suburban: ADT ≤ 16000 ADT > 16000	 10-ft 20-ft	
	Urban: Curbed (DS ≥ 50 MPH) Uncurbed (DS ≥ 50 MPH)	 Use Suburban Criteria	
	Rural Town, Urban, Urban Core: Curbed (DS ≤ 45 MPH) Uncurbed (DS ≤ 45 MPH)	 4-ft from FOC 10-ft	
Ramps and Direct Connectors	Rural Suburban Urban Core Urban	16-ft	TxDOT Criteria
Collector-Distributor Roads	Rural Suburban Urban Core Urban	16-ft	TxDOT Criteria
Frontage Roads	Use Collector Criteria		
Notes:			
1. "DS" stands for Design Speed			

Table B-8: 4R Lateral Offset to Obstructions (Minimum)

Facility Type	Additional Factors	Offset	Reference
Freeway Arterials Collectors Local	All Contexts: Curb[1] Barrier[2]	 1.5-ft 1-ft	TxDOT Criteria
Notes:			
1. Measured from nominal face of curb.			
2. Measured from toe of barrier.			

Table B-9: 4R Railroad Overpass Geometrics

Facility Type	Reference
Freeway Arterial Collector Local	TxDOT Bridge Project Development Manual

Table B-10: 4R Sag Vertical Curve Length[1]

Facility Type	Reference
Freeway Arterial Collector Local	TxDOT Criteria See Figure 4-7 & Table 4-12
Notes:	
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 correlations with the k-value. If the minimum k-value is satisfied for a sag vertical curve (Figure 4-7), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings, see Figure 4-29. See 4.11.5 for additional guidance on comfort criteria.	

Table B-11: 4R Superelevation (Non-rate elements)

Facility Type	Element	Reference
Freeway Arterial Collector Local	Superelevation Transition Length	TxDOT Criteria See Chapter 4.7.6
Freeway Arterial Collector Local	Superelevation Transition Location	TxDOT Criteria See Chapter 4.7.7

Table B-12: 4R Guardrail Length[1]

Facility Type	Reference
Freeway Arterial Collector Local	TxDOT Criteria See Chapter 17.3- Longitudinal Barriers
Notes:	
1. Unless for access accommodation.	

Table B-13: TxDOT's 4R Non-Controlling Criteria for Bicycle Facilities (Minimum)

Facility Type	Facility Type	Criteria	Reference
Arterials Collectors Local	Shared Use Path Width	10-ft[2]	See RDM chapter 18 section 18.4.1 for further guidance
	Separated Bike Lane/Buffered Bike Lane Width DS ≤45mph DS >45mph	 ----- -----	See RDM chapter 18 section 18.4.2 for further guidance
	Bike Accessible Shoulder Width DS ≤45mph[1] DS >45mph[1] Shoulders adjacent to bridge rail, MBGF, and other vertical elements	 4-ft 5-ft 5-ft	See RDM chapter 18 section 18.4.6 for further guidance

Notes:

1. DS stands for design speed

2. In rare cases meeting the following conditions 8-ft minimum may be used.
- Bicycle traffic is expected to be low < 50 bicyclist/hour
- Pedestrian use of the facility is not expected to be more then occasional or less than 30% of total path traffic
- Horizontal and vertical alignments provide frequent passing and resting opportunities where the width is at least 10-ft
- The path will not be regularly subjected to maintenance vehicle loading conditions that would cause pavement edge damage

Table B-14: 3R Design Speed (Minimum)[1]

Facility Type	Additional Factors	Design Speed	Reference
Freeway	----	----	See Table A-1 for 4R Criteria in Appendix A
Arterials Collector Local	Rural: 2-Lane 4-Lane Undivided 4-Lane Divided 6-Lane Divided	 30 MPH 50 MPH 50 MPH 50 MPH	TxDOT Criteria
	Rural Town Suburban Urban Core Urban	30 MPH	
Frontage Roads	Rural	30 MPH	TxDOT Criteria
	Suburban Urban	30 MPH	
Notes:			
1. Applicable to low volume (ADT < 1500) roadways.			

Table B-15: 3R Horizontal Curve Radius (Minimum)[1]

Facility Type	Superelevation, e-max	Curve Radius (feet) for Design Speed (mph)															Reference
		15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Low-speed Rural Town, Suburban, urban and Urban Core Facilities	0.04	--	--	--	--	--	--	--	---	---	---	---	---	---	---	---	See Table A-4 for 4R Criteria in Appendix A
	NC	--	--	--	--	--	--	--									
High-speed Suburban and Urban Facilities	0.04	---	---	---	---	---	---	---	--	--	--	---	---	---	---	---	See Table A-4 for 4R Criteria in Appendix A
Rural and High-speed Suburban, urban and Urban Core Facilities	0.06	--	--	--	--	--	--	--	--	--	--	---	--	--	--	---	See Table A-4 for 4R Criteria
	0.08	--	--	--	--	--	--	--	--	--	--	--	--	--	--	---	

Notes:

1. Applicable to low volume (ADT < 1500) roadways.

Table B-16: 3R Stopping Sight Distance (SSD) (Minimum)[1]

Facility Type	Design Speed (mph)															Reference
	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Stopping Sight Distance (feet) for Design	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	See Table A-6 for 4R Criteria in Appendix A
Notes:																
1. Applicable to low volume (ADT< 1500) roadways.																

Table B-17: 3R Maximum Grade[1]

Facility Type	Additional Factors	Terrain	Minimum Grade (%) for Design Speed (mph)															Reference
			15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	
Freeway	Rural and Urban	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria in Appendix A
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
Arterial	Rural	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria in Appendix A
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
	Urban	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria in Appendix A
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
Collector	Rural	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria in Appendix A
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
	Urban	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria in Appendix A
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
Local	Rural	Level	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria in Appendix A
		Rolling	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
	Rural Town, Suburban, Urban, Urban Core	All	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	See Table A-6 for 4R Criteria in Appendix A
													---	---	---	---	---	

Notes:

1. Applicable to low volume (ADT < 1500) roadways

Table B-18: 3R Superelevation (Non-rate elements)[1]

Facility Type	Element	Reference
Freeway Arterial Collector Local	Superelevation Transition Length	TxDOT Criteria See Chapter 4.7.6
Freeway Arterial Collector Local	Superelevation Transition Location	TxDOT Criteria See Chapter 4.7.7
Notes:		
1. Applicable to low and high volume roadways.		

Table B-19: Bridge Width[1]

Facility Type	Reference
Freeway Arterial Collector Local	See TxDOT Bridge Project Development Manual
Notes:	
1. Applicable to low and high volume roadways.	

Table B-20: Deficient Bridge Rail[1]

Facility Type	Reference
Freeway Arterial Collector Local	See TxDOT Bridge Railing Manual
Notes:	
1. Applicable to low volume (ADT < 1500) roadways	

Table B-21: 3R Clear Zone (Minimum)[1]

Facility Type	Additional Factors	Clear Zone	Reference
Freeways	---------- ---------- ----------	----------	See 4R criteria values
Arterials Collectors Local	Rural: 2-Lane Current ADT < 1500 Current ADT ≥ 1500	 7-ft 10-ft	TxDOT Criteria
	4-Lane Undivided 4-Lane Divided 6-Lane Divided	16-ft	
	Rural Town Suburban Urban Core Urban	To back of curb or outside edge of shoulder	TxDOT Criteria
Frontage Roads	Rural: Current ADT < 1500 Current ADT ≥ 1500	 7-ft 16-ft	TxDOT Criteria
	Suburban Urban	To back of curb or outside edge of shoulder	
Notes:			
1. Applicable to low volume and high volume roadways.			

Table B-22: 3R Speed Change Lane Width (Minimum)[1]

Facility Type	Additional Factors	Turn Lane Width	Reference
Freeways	--------- --------- --------	See 4R criteria values	
Arterials and Collectors	Rural: 2-Lane 4-Lane Undivided 4-Lane Divided 6-Lane Divided	10-ft	TxDOT Criteria
	Rural Town Suburban Urban Core Urban	10-ft	TxDOT Criteria
Local	Rural Town Suburban Urban Core Urban	10-ft	TxDOT Criteria
Notes:			
1. Applicable to low volume and high volume roadways.			

Table B-23: 3R Length of Speed Change Lanes (Minimum)[1]

Facility Type	Reference
Arterials and Collectors	TxDOT Criteria See 4.10.2 and Table 4-14
Notes:	
1. Applicable to low volume and high volume roadways.	

Table B-24: 3R Curbside Parking Lane Width (Minimum)[1]

Facility Type	Additional Factors	Parking Lane Width	Reference
Arterials & Collectors	Rural Town Urban Core Urban	7-ft[2]	TxDOT Criteria
Notes:			
1. Applicable to low volume and high volume roadways.			
2. In commercial and industrial areas an 8-ft minimum width should be used.			

Table B-25: 3R Guardrail Length[1,2]

Facility Type	Reference
Freeway Arterial Collector Local	TxDOT Criteria See Chapter 17.3 - Longitudinal Barriers
Notes:	
1. Applicable to low volume and high volume roadways.	
2. Unless for access accommodation.	

Table B-26: 3R Sag Vertical Curve Length[1]

Facility Type	Reference
Freeway Arterial Collector Local	TxDOT Criteria See Figure 4-7 & Table 4-12
Notes:	
1. Applicable to low volume and high volume roadways.	

Appendix C
List of References

(Note for other official TxDOT Manuals, Guides (e.g., TMUTCD, Bridge Railing Manual, Sign Crew Field Book) the currently adopted version is applicable unless otherwise stated.)

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