Manual Notice  2020-1

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Purpose

The Roadway Design Manual has been revised to update roadway policy in accordance with FHWA guidelines and with comments received from Design Division, other Divisions, and TxDOT's 25 Districts. As a result, revisions were made to Chapters 1-8 and Appendices A, B and C. Also, new Appendices D, and E were added to the manual, that is, Right Turn Slip Lane Design Guidelines and Alternative Intersections and Interchanges, respectively. Likewise, minor errata type corrections and edits were made throughout the manual.

Contents

Chapter 1

Section 1:

◆ Revised Roadway Design Manual Format for chapters 7, 8, Appendix C, and to include the addition of Appendices D, and E.

◆ Under External Reference Documents, updated references.

Section 2:

◆ Revised controlling criteria per FHWA guidance for Design Exceptions, Waivers, and Variances. Reduced controlling criteria from 13 to 10.

◆ Under Design Exceptions, added guidance that 4' minimum width is required for new bike lanes. Widths less than 4' will require a design exception.

◆ Under Design Exceptions, added guidance that design exceptions for bridge rails shall be sent to Bridge Division.

◆ Under Design Waivers, removed "non-controlling criteria for the" from introductory sentence.

◆ Under Design Waivers, removed requirement for copy of documentation being furnished to Design Division.

◆ Under Design Waivers, added "Shared Use Paths (if this is the chosen Bicycle/Ped facility)" to 4R.
Under Special Facilities, moved sentence "Design waivers are not applicable..." to the bottom of the section.

Revised Design Exceptions and Design Waivers for Bicycle Facilities.

Revised section on Design Variances for design documentation for PROWAG.

Under THFN, provided link to Design Division’s Roadway and Hydraulics website.

Section 3:

Revised entire section of Schematic Layouts to include additional items for proper review and evaluation.

Added "Location of wildlife crossing structures" to list.

Section 4:

(IAJR) was revised to incorporate: FHWA Updated Policy on Access to the Interstate System, May 22, 2017 and Current TxDOT IAJR Policy Memo, October 2018.

Removed reference to 23 CFR 630 and replaced it with reference to United States Code (USC)-Title 23 Section 111.

Added reference to the TxDOT IAJR SOP.

Section 5:

Replaced the Preliminary Design Submission Table with a list of preliminary design processes that affect/influence decisions and a list of reference material for the various submittal processes.

Section 6:

Added verbiage for twin bridge structures providing 10' distance to facilitate access for inspections.

Chapter 2

Section 2:

Under Speed, added guidance on "Speed, Design Speed, Operating Speed, Posted Speed, and Running Speed".

Added guidance and discussion on "Terrain".

Added content to emphasize Crash Data, Highway Safety and Safety Analysis.

Updated Table 2-2. Defined variable (t) as time in seconds.

Section 3:
Under Intersection Sight Distance, added guidance that intersections should be free of conflict points to the extent allowable to promote safety.

Under Intersection Sight Distance, added reference to AASHTO for intersection control guidance.

Added new Figure 2-1.

Section 4:

Updated Table 2-3. Changed title of table and changed footnote 2.

Updated Table 2-4. Changed title of table and deleted footnote 1.

Updated Table 2-6. Changed title of table.

Updated Table 2-7. Changed title of table.

Specified that the calculated transition length, LCT, can be calculated using adjustment factors as shown in Table 2-9.

Under Super Elevation Transition Type, specified that linear superelevation transition may be used.

Section 5:

Clarified that the "roadway" grade should seldom be less than 0.5 percent for unpaved ditches and 0.25 percent for lined channels.

Combined content for Crest and Sag Vertical Curves into one section named "Vertical Curves".

Added guidance on Sag Vertical Curves discussing the 4 different criteria, light beam distance, and stopping site distance.

Replaced Figure 2-6.

Replaced Figure 2-7.

Section 6:

Added cross slope guidance stating that the cross slope should not exceed 3 percent on a tangent alignment unless there are 3 or more lanes in one direction of travel.

Under Pavement Cross Slope changed guidance for the algebraic difference of cross slope between the traveled way and the shoulder grades should not exceed 6 percent. (Previously 6 to 7 percent)

Clarified the width of flush or curbed medians for pedestrian refuge is 6' from BOC to BOC preferred and 6' from FOC to FOC minimum.

Under Medians, added content for Design Guidance to Reduce Consequences of Median Encroachments, Design Guidance to Reduce likelihood of Median Encroachments, and Countermeasures to Reduce Likelihood of Median Encroachments.
Guidance was added for bicycle accommodation for urban streets and rural projects.

Guidance was added for measurement of shoulder widths on bridge structures.

Under Shoulder Widths, adjusted language and guidance for Shoulder Widths.

A clarification was made for shoulder widths to accommodate bicycle facilities across bridges being replaced or rehabilitated to be 5' minimum unless it's on the off-system and less than 400 ADT.

Under Sidewalk and Pedestrian Elements, requirements under PROWAG and TAS was added.

Under Sidewalk Location, removed the guidance for roadways classified as rural, the shoulder may be used to accommodate pedestrian and bicycle traffic. Also, removed guidance for where a shoulder serves as part of the pedestrian access route, it must meet PROWAG requirements.

Under Sidewalk Width, added guidance that where there is a 4' sidewalk for short distances, a 5' width is required just after a pedestrian ramp for wheel chair maneuverability.

Under Walking Speeds, added "or near schools" for a location of walking speed of 3.0 ft/s.

Under Street Crossings, added guidance for when push buttons are required for crossings.

Updated Figure 2-8. Changed Figure to black & white.

Updated Figure 2-10. Changed Figure to black & white.

Under Curb Ramps and Landings, changed "should" to "shall" for guidance on curb ramps and landings

Under Curb Ramps and Landings, added "Curb inlets" to list of obstructions that should not be located within the curb ramp, maneuvering area, or landing.

Under Street Furniture, added guidance with PROWAG & TAS.

Under Front Slope, added guidance for slopes greater than 1V:2.5H.

Under Clear Zone, deleted sentence "For fill slopes steeper than 1V:4H, errant vehicles have a reduced chance of recovery and the lateral extent of each roadside encroachment increases."

Table 2-12 was updated. Changed clear zone criteria for minimum and desirable for Rural Arterial.

Section 7:

Under Bridge Class Drainage Culverts, clarified language for bridge class culvert protection.

Updated Table 2-14. Added verbiage to table under "Treatment".

Updated Table 2-14 to include Note to refer to Bridge Division for further guidance.

Updated Figure 2-11. Changed "Note" on Figure.

Under Side Ditches, changed "horizontal clearance" to clear zone. Also, made this change for the entire RDM.
Section 8:

- Clarified and expanded on requirements for a non-state intersecting roadway’s need to adapt its alignment to a TxDOT road. Also, added guidance for an intersecting road that is modified and changed to tie into a TxDOT facility due to the request of the county or city.

Chapter 3

Section 2:

- Table 3-1, Maximum Gradient link was corrected to Table 2-11.
- Table 3-1, footnote 11 was changed to "A 5' minimum clear space for bicycles should be provided on bridges being replaced or rehabilitated except on off-system facilities with less than 400 ADT."
- Table 3-1 footnote 12 was added giving guidance on vertical clearance requirements for roadways on the THFN.
- Under Raised Medians, guidance was added stating "A median width of 18' is required if pedestrian refuge area is needed, see Figure 2-10."
- Under Raised Medians, changed guidance to 6 feet for a pedestrian divider.
- Under Left-Turn Deceleration Lanes, guidance was added stating "Where pedestrians may be present, the divider must be a minimum of 6', see Figure 2-10."
- Updated Table 3-3. Changed footnote 5.
- Updated Table 3-4. Added more "Speeds" to the table.
- Added guidance for Right-Turn Acceleration Lanes stating "Acceleration lanes typically are not used on urban streets. See Section 5, Figure 3-10 for acceleration distances and taper lengths, if an acceleration lane may be necessary."

Section 3:

- Updated Table 3-5. Added footnotes 7 & 8 to table and fixed reference links.
- Under Clear Zones, link for Clear Zones was corrected to Table 2-12.

Section 4:

- Updated Table 3-6. Changed footnotes 1 & 2 and corrected superscripts on table.
- Updated Table 3-7. Corrected footnote 2 and reference links.
- Updated Table 3-8. Added footnote 7 and corrected superscripts on table.
- Updated Table 3-9. Footnote 1 was adjusted to add guidance on the Clear Width on bridge structures.
- Updated Table 3-10. Table was reformatted.
Updated reference to Table 3-11.
Under Left-Turn Deceleration Lanes, corrected table references stating "Lengths of left-turn deceleration lanes are provided in Table 3-11 for Two-Lane Highways and Table 3-13 for Multi-Lane Rural Highways."
Under Right-Turn Deceleration Lanes, added guidance stating "Where the right turn deceleration or acceleration lane is being constructed adjacent to the through lanes, the minimum land width is 10' with a 2' surfaced shoulder."
Under Right-Turn Acceleration Lanes, corrected table reference to Table 3-3 for acceleration distances and taper lengths.
Under Intersections, added guidance stating that "Desirably, the roadways should intersect at approximately right angles, and should not intersect less than 75 degrees. Where crossroad skew is flatter than 75 degrees to the highway, the crossroad should be re-aligned to provide for a near perpendicular crossing."

Section 5:
Table 3-12, for Design Speed (Arterials) changed footnote to 1.
Table 3-12, added guidance for Vertical Clearance for new structures.
Table 3-12, footnotes 2 & 3 clarified what heavy betterment and unusual circumstances means.
Table 3-12, added footnote 5 for guidance on roadways on the THFN.
Table 3-12, added footnote 6 for design guidance on vertical clearance.
Table 3-12, re-wrote footnotes 2 & 3 to include "documented through design exception" and deleted example.
Table 3-12, changed "flat" to "level".
Table 3-12, added 3 columns of 12' for Lane Width. Updated Metric.
Table 3-12, clarified language for bridge clear width in footnote 7.
Under Four-Lane Undivided Highways, revised paragraph for better understanding of previous verbiage of "betterment projects".
Corrected link to Figure 2-2, Determination of Length of Superelevation Transition.
Corrected link to Table 2-11, Maximum Grades.
Corrected link to Table 8-11, Earth Fill Slope Rates.
Updated Table 3-14. Table was reformatted.

Section 6:
Table 3-15, corrected links to Clear Zone Table 2-12 and Maximum Grades Table 2-11.
Table 3-15, added row for Lane and Shoulder Widths.
Under Level of Service, added guidance for other Measure of Effectiveness (MOE’s) may include travel time, speed and queue lengths in heavily congested areas.

Under Lane Width and Number, corrected reference to Figure 3-15, Typical Freeway Sections.

Under Shoulders, corrected reference to Figure 3-15, Typical Freeway Sections.

Under Medians, corrected reference to Figure 3-15, Typical Freeway Sections.

Under Vertical and Clear Zone at Structures, added guidance for roadways on the THFN.

Under Vertical and Clear Zone at Structures, added guidance to refer to specific railroad company guidelines for additional vertical clearance requirements.

Under Vertical and Clear Zone at Structures, added railroad guidance.

Under Vertical and Clear Zone at Structures, removed sentence on the exceedance of 3 inches which was obsolete due to the requirements of the THFN.

Removed Figure 3-16, Typical Highway Railway Overpass and all its references.

Under Horizontal, corrected link to Table 2-12, Clear Zones.

Updated Table 3-18. Corrected superscripts on table.

Under Frontage Road Design Criteria, corrected link to Table 2-12, Clear Zones.

Updated Table 3-19. Table was reformatted.

Under Design Speed, added guidance for design speed for where a ramp joins a frontage road.

Under Horizontal Geometrics, corrected links to Figures 3-28, 3-29, 3-30, 3-31, 3-32, 3-33, and 3-34.

Updated Figure 3-36. Changed weaving distance to be from nose of gore to nose of gore.

Updated Table 3-21. Table was reformatted.

Under Grades and Profiles, guidance was added for ramp grades stating "For special circumstances and topography conditions, the following grades may be used: Ramp Design Speed 25-30 mph-7% max, 35-40 mph-6% max, and 45 mph or greater-5% max."

Section 8:

Corrected title of Section 8 to read Texas Highway Freight Network (THFN).

Chapter 4

Section 2:

Under Geometric Design, added language for Bridge Width for clarification.

Updated Table 4-1. Added footnote "e", changed Horizontal Clearance to Clear Zone, and changed title of table.
Updated Table 4-2. Changed title of table, changed Horizontal Clearance to Clear Zone, and added footnote f.

Table 4-2, changed 16' clear zone to 10' clear zone for consistency with 4R clear zone table.

Updated Table 4-3. Changed title of table and changed Horizontal Clearance to Clear Zone.

Under Alignment, added guidance on what is considered a 3R project and a 4R project.

Under Alignment, added guidance on Design Exceptions for 3R projects.

Section 3:

Under Safety Design, replaced "accident" with "crash".

Under Safety Design added guidance and bullet item for Crash Analysis.

Under Safety Design, added language for wildlife crashes to section.

Under Safety Design, added guidance for High Friction Surface Treatment (HFST) to address deficiencies in curve radius or superelevation.


Section 4:

Updated Table 4-4. Corrected Bridge Width to be retained from 22' to 24'.

Updated Table 4-4. Corrected superscript on table.

Chapter 6

Section 3:

Under Working Agreements, added guidance stating that "TxDOT design standards are used on Park Roads (PR) that lead or enter a state park and are designated on the State Highway System. Parks and Wildlife Roads (PW) will use the standards of Texas Parks and Wildlife Department standards."

Section 4:

Under Guidance for Bicycle Facilities, changed "bicycle paths" to "shared use paths".

Under Guidance for Bicycle Facilities, added guidance for what is a bicycle lane, shared lane, and shared use path.

Under Guidance for Bicycle Facilities, guidance was added for the preferred order for design is to have a shared use path, then a bike lane, then a shared lane.
Under Guidance for Bicycle Facilities, added guidance that shared lanes are not appropriate on high speed facilities and should be avoided.

Under Guidance for Bicycle Facilities, added guidance for a striped bicycle lane the clear width is 4' minimum and 5' desirable.

Under Design Exceptions and Design Waivers for Bicycle Facilities added new guidance for design exceptions and design waivers for bicycle lanes, shared lanes and shared use paths.

Chapter 7

Section 1:
- Renamed Section 1 to Longitudinal Barriers and Roadside Safety Hardware Criteria.
- Under Guardrail, added guidance stating that "Guardrail should be offset at least 4.0' and desirably 5' or more from the nearest edge of fixed objects."
- Under Crash Cushion Categories, added additional guidance on crash cushions.
- Under Roadside Safety Hardware Criteria, added language on MASH criteria and guidance.

Section 3:
- Under Undercrossings, added guidance to provide ample drainage.

Section 5:
- Removed "Shoulder Texturing" from title and replaced it with "Rumble Strips" in Section 5, and updated guidance on rumble strips to make consistent with Traffic Safety Division RS standards, and FHWA guidance.

Section 7:
- Updated Table 7-2 and changed Radii for 3-Centered Compound Curve, Symmetric for a 75 degree angle/Passenger vehicle (P) to 100-25-100.

Chapter 8

Section 2:
- Updated Table 8-1. Table was reformatted.
- Updated Table 8-2. Table was reformatted.
- Updated Table 8-3. Table was reformatted.
- Updated Table 8-5. Table was reformatted.
- Updated Table 8-6. Table was reformatted.
- Updated Table 8-7. Table was reformatted.
Under Superelevation, added guidance for superelevation on bridge structures to begin/end superelevation transition at bridge bent line.

Updated Table 8-8. Table was reformatted.

Updated Table 8-9. Table was reformatted.

Section 3:

Changed "Horizontal Clearance to Clear Zone".

Updated Table 8-10. Changed title of table and reformatted table.

Updated Table 8-11. Table was reformatted.

Section 4:

Updated Table 8-12. Table was reformatted.

Updated Table 8-13. Table was reformatted.

Appendix A – Longitudinal Barriers

Section 2:

Added verbiage giving an overview of Types of Barrier. Added verbiage with illustration defining Working width.

Section 3:

Under Rail Element and Blockouts paragraph, added verbiage clarifying rail height and blockout applications.

Section 4:

Updated deflection considerations for guard fence; updated Fig.A-6 accordingly.

Section 6:

Updated Length of Need Fig-A-8 for additional clarity; added verbiage for the required protection of bridge class culverts.

Updated Table A-2.

Section 8:

Under Application, for Width of the median, added "(measured from edge of pavement to edge of pavement)."

Added additional Cable Median Barrier guidance with respect to placement and appropriate applications. New Fig.A-16.
Section 9:

- Updated Fig. A-17 & A-18 to provide additional overlap for MBGF and cable barrier applications.

Appendix B – Treatment of Pavement Drop-offs in Work Zones

General:

- Minor errata updates.

Section 1:

- Table for Treatment Guidelines for Pavement Drop-offs in Construction Work Zones, update Note 3.

Appendix C – Driveway Design Guidelines

Section 1:

- Added guidance for support documentation in the form of traffic operations and safety analysis.

Section 2:

- Clarified definition of driveway and commercial driveway.

Section 3:

- Clarified pedestrian application guidance at a driveway. Updated Table C-2 and Table C-3 to clarify guidance on divider width in cases of a pedestrian refuge.
- Updated Table C-1. Changed title of table and added 15' Min. to table.
- Updated Table C-2. Changed footnotes 1 & 2 and added "Min." to table under Radius.
- Updated Table C-3 footnote 2.
- Under Divided Driveways, added "...and 6 feet for a Ped. Refuge." For a minimum width of a slightly raised divider.

Section 6:

- Under General Guidelines, changed the minimum refuge area to 5 x 6 feet.

Section 8:

- Updated references.

Appendix D – Right-Turn Slip Lane Design Guidelines

New Appendix providing guidance on the design of right-turn slip lanes.
Appendix E – Alternative Intersections and Interchanges

New Appendix providing guidance on the alternative intersection and interchange forms.

Instructions

This manual, and all revisions, applies to all highway and street project development, whether developed by the department or with consultant staff. Due to projects that may be further along in development with current criteria, this manual, and all revisions, will be effective for all projects beginning with the January 2021 letting. Project development using this manual and its revisions prior to that date is at the option of the district.

Contact

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

Archives

Past manual notices are available in a pdf archive.
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Preface

Non-discrimination

Texas Department of Transportation (TxDOT) policy is to ensure that no person shall 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

The *Roadway Design Manual* was developed by TxDOT to provide guidance in the geometric design of roadway facilities. It should be noted at the outset that this document is a guide containing geometric design recommendations and does not represent an absolute design requirement.

The *Roadway Design Manual* represents a synthesis of current information and operating practices related to the geometric design of roadway facilities. The fact that updated design values are presented in this document does not imply that existing facilities are unsafe. Nor should the publication of updated design guidelines 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 such stakeholders as 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 not only provide for beneficial design components, but also ultimately provide the most beneficial total roadway system of which each individual design project is only a part.

While much of the material in the *Roadway Design Manual* can be considered universal in most geometric design applications, there are many areas that 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 made on the basis of an engineering study of the location, operational experience, and objective analysis. Thus, while this document provides guidance for the geometric design of highways and streets, it is not a substitute for engineering judgment. Further, while it is the intent that this document provide geometric design guidance, the *Roadway Design Manual* does not represent a legal requirement for roadway design.

Roadway design is a continually evolving process. As additional information becomes available through experience, research, and/or in-service evaluation, this guide will be updated to reflect current state-of-the-practice geometric design guidance for roadway facilities.
Chapter 1 — Design General

Contents:

Section 1 — Overview
Section 2 — Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight Network (THFN) Design Deviations
Section 3 — Schematic Layouts
Section 4 — Access to the Interstate System
Section 5 — Preliminary Design
Section 6 — Maintenance Considerations in Design
Section 1 — Overview

Application of Design Guidelines

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

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

It should be noted that roadway design criteria and technology is a rapidly changing field of study. The fact that new design values are presented or updated herein does not imply that existing highway conditions are less safe. Also, continually enhanced design practices do not mandate the need for improvement projects. With a significant transportation infrastructure in place, the intention is to use the most current design techniques on projects scheduled for future construction. The manual is intended to result in projects, which provide user safety and operational efficiency while taking into account environmental quality. Various environmental impacts can be mitigated or eliminated by the use of appropriate design practices. To the extent practical, the selection of cost effective design criteria can allow the finished project to be more consistent with surrounding terrain and/or settings.

Roadway Design Manual Format

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

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

Chapter 3 describes new location and reconstruction (4R) project design criteria. These projects usually represent the highest type design since these are either new roadways or almost totally reconstructed roadway sections. This chapter of the RDM is broken into roadway classifications such as urban streets, suburban roadways, two-lane highways, multi-lane rural highways, and freeways.

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

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

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

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

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

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

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

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

Appendix D provides guidance on right-turn slip lanes.

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

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

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

AASHTO has established various policies, standards, and guides relating to transportation design practices. These documents are approved references to be used in conjunction with the RDW. However, the instructions given in the RDM will take precedence over AASHTO documents unless specifically noted otherwise.
Section 2 — Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight Network (THFN) Design Deviations

Overview

This subsection discusses the following topics:

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

Design Exceptions

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

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

Design exceptions involving the design loading structural capacity or bridge rails shall be sent to the Bridge Division for their review and approval.
All design exceptions must be signed by the TxDOT District Engineer and this signature authority cannot be delegated. Design exceptions must be documented on the Form 1002.

For flexibility and efficiency in meeting project design schedules, the review of design exceptions and recommendations for approval/non-approval may be established individually by each district. For example, a four person review committee might be established which includes:

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

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

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

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

**New Location and Reconstruction Projects (4R).** The list below gives the controlling criteria for 4R projects that will require a design exception.

- Design Speed
- Lane Width
- Shoulder Width
- Horizontal Curve Radius
- Superelevation (Rate only)
- Stopping Sight Distance (SSD) [see Note below]
- Maximum Grade
- Cross Slope
- Vertical Clearance
- Design Loading Structural Capacity

**NOTE:** 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
Resurfacing, Restoration or Rehabilitation (3R) Projects. The list below gives the controlling criteria for 3R projects that will require a design exception. For 3R projects, high volume roadways are defined as current ADT of 1500 and greater.

- Deficient Bridge Rails (high volume roadways)
- Design Speed (high volume roadways)
- Horizontal Curve Radius (high volume roadways)
- Superelevation Rate only (high volume roadways)
- Maximum Grade (high volume roadways)
- Stopping Sight Distance (SSD) (high volume roadways) [See Note below]
- Lane Width
- Shoulder Width
- Design Loading Structural Capacity (see Bridge Project Development Manual)

NOTE: SSD only applies to crest vertical curves for the purposes of a Design Exception for 3R high-volume roadways. SSD for crest vertical curves is a direct correlation with the K-Value. If the minimum K-Value is satisfied for a crest vertical curve (Fig. 2-5), then the vertical SSD is satisfied under usual conditions.

Resurfacing or Restoration Projects (2R). Design exceptions are required for 2R projects any time the existing geometric or bridge features for the proposed project will be reduced.

Bicycle Facilities. If the minimum requirements given in the AASHTO Guide for the Development of Bicycle Facilities for bicycle lanes or shared lanes cannot be met, a design exception will need to be submitted. See Chapter 6, Section 4 for additional information.

- For new shared lanes, the minimum lane width shall be 14 ft [4.2 m]. Widths less than this will require a design exception.
- For new bicycle lanes, the minimum clear width shall be 4 ft [1.2 m] measured to the longitudinal joint. Widths less than this will require a design exception.
- If the minimum requirements given in the AASHTO Guide for the Development of Bicycle Facilities for shared use paths cannot
be met, a design waiver will be necessary per the following Design Waiver section.

Special Facilities. For off-system bridge replacement and rehabilitation projects with current ADT of 400 or less, the following design elements must meet or improve conditions that are typical on the remainder of the roadway or a design exception will be necessary:

- Design Speed
- Lane Width
- Shoulder Width
- Design Loading Structural Capacity (see Bridge Project Development Manual)
- Horizontal Curve Radius
- Maximum Grade
- Cross Slope
- Superelevation (Rate Only)
- Stopping Sight Distance (SSD) [See Note below]

NOTE: SSD only applies to crest vertical curves for the purposes of a Design Exception for off-system bridge projects. SSD for crest vertical curves is a direct correlation with the K-Value. If the minimum K-Value is satisfied for a crest vertical curve (Fig. 2-5), then the vertical SSD is satisfied under usual conditions.

Off-System Historically Significant Bridge Projects. The list below gives the controlling criteria that will require a design exception.

- Roadway Width
- Load Carrying Capacity (Operating Rating)

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

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

When the criterion is not met in a non-controlling category, a design exception is not required. However, a departure from the minimum criteria will be handled by design waivers at the district level. Design waivers will be granted as the district authorizes. The complete documentation should be retained permanently in the district project.

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

**New Location and Reconstruction Projects (4R).**

- Curb Parking Lane Width
- Speed Change (refuge) Lane Width
- Length of Speed Change Lanes
- Curb Offset
- Bridge Width (See Bridge Project Development Manual), Ch. 3- Sec. 1
- Median Opening Width
- Clear Zone
- Lateral Offset to Obstructions
- Railroad Overpass Geometrics
- Sag Vertical Curve Length [See Note below]
- Superelevation (Non-rate elements)
- Guardrail Length (unless for access accommodation; see Appendix A, Metal Beam Guardrails).
- Shared Use Paths (if this is the chosen Bicycle/Ped facility)

**NOTE:** 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 (Fig. 2-6), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings see Fig. 2-1A.

**Resurfacing, Restoration or Rehabilitation (3R) Projects.** For 3R projects, low volume roadways are defined as a current ADT of less than 1500.

- Design Speed (low volume roadways)
Chapter 1 — Design General

Section 2 — Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight

- Horizontal Curve Radius (low volume roadways)
- Stopping Sight Distance (SSD) (low volume roadways) [See Note 1 below]
- Superelevation Rate only (low volume roadways)
- Superelevation (Non-rate elements)
- Maximum Grade (low volume roadways)
- Bridge Width (See Bridge Project Development Manual), Ch. 3-Sec. 1
- Deficient Bridge Rails (low volume roadways)
- Clear Zone
- Turn Lane Width
- Length of Speed Change Lanes
- Parallel Parking Lane Width
- Guardrail Length (unless for access accommodation; see Appendix A, Metal Beam Guardrails)
- Sag Vertical Curve Length [See Note 2 below]

**NOTE 1:** SSD applies only 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 (Fig. 2-5), then the vertical SSD is satisfied under usual conditions.

**NOTE 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 (Fig. 2-6), then the Sag Vertical Curve Length is satisfied under usual conditions. For minimum Sag Vertical Curve Lengths at Under-Crossings see Fig. 2-1A.

**Resurfacing or Restoration Projects (2R).** Design waivers are not applicable to 2R projects.

**Special Facilities.** For off-system bridge replacement and rehabilitation projects:
- Superelevation (Non-rate elements)
- Sag Vertical Curve Length [See Note below]
- Bridge Width (Structure Width, Nominal Face to Face of Rail: 24 ft [7.2 m]).
NOTE: Sag Vertical Curve Length applies to sag vertical curves for the purposes of a 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 (Fig. 2-6), then the Sag Vertical Curve Length is satisfied under usual conditions.

Design waivers are also necessary when the minimum requirements given in the AASHTO Guide for the Development of Bicycle Facilities for shared use paths cannot be met.

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

Design Variances

A design variance is required whenever the design guidelines specified in the Texas Accessibility Standards (TAS) are not met. Design variances shall be sent to Texas Department of Licensing and Regulation (TDLR) for approval. Refer to Sidewalks and Pedestrian Elements in Chapter 2 for additional discussion. Design variance documentation should also be developed when the Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right of Way (PROWAG) cannot be met. This documentation should be kept in the project files.

Texas Highway Freight Network (THFN) Design Deviations

A Texas Highway Freight Network (THFN) Design Deviation will be required for projects Letting September 1, 2020 or later that do not meet specified Bridge Vertical Clearance Requirements. Chapter 3, Section 8 contains the specific requirements for the THFN. The Deviation request is exclusive of (and in addition to) the Design Exception that may be needed for Vertical Clearances not meeting the specified requirements in Chapter 3 of the respective roadway facility. The THFN Deviation requests will be submitted to the Design Division – Project Development Support Section, and will be reviewed by an assigned Design Deviation Committee.

A Deviation request form, process flowchart, and other information can be found on the Design Division – Roadway and Hydraulics Design website.
Section 3 — Schematic Layouts

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

- A Design Summary Report (DSR) appropriate for the stage of the project development
- General project information, including TxDOT District, County, Control-Section-Job (CSJ), roadway name, project limits, project length, stationing equations, project location map, design speed, functional classification, and existing and projected ADT volumes
- TxDOT logo and copyright, the Engineer's firm name (if applicable), the Preliminary Engineer's Seal, the percent complete, and the date of latest revision
- North arrow, legend, drawing scales, and labeling for the beginning and ending of the project
- Traffic diagram(s) showing existing and projected ADT volumes of main lanes, ramps, frontage roads, turnarounds, and cross roads, and the breakdown of those volumes for turning movements, as applicable
- Proposed horizontal and vertical alignments for proposed main lanes, ramps, frontage roads, ancillary roads, and cross roads, including labels for all curve data, elevations, grades, and minimum vertical clearances over highways. Superelevation rates, locations, and transition lengths
- Existing and proposed bicycle and pedestrian accommodations, including curb ramp locations and roadway crossing locations
- Existing and proposed horizontal alignments for railroads
- Existing roadways in plan-view and existing ground profiles and elevations along each proposed alignment
- Existing vertical alignments for cross roads to remain in place at proposed interchanges or grade separations
- Pavement marking and arrows indicating the number of lanes, direction of travel, and lengths of lane/shoulder tapers and weaving areas
- Aerial imagery or an orthophotograph in the plan view, if available
- Locations and widths of medians and median openings
- Lengths and geometrics of speed change lanes, turn lanes, auxiliary lanes, and storage bays
- Portions of existing roadways and structures to be removed or closed
- Existing and proposed right of way (ROW) limits, property lines, and driveways
- Existing and proposed control-of-access (COA) lines
Chapter 1 — Design General

Section 3 — Schematic Layouts

- Relevant affected utilities
- Existing and proposed bridges and bridge class culverts, including the beginning and ending stations of each proposed structure
- Locations of retaining walls and/or noise walls
- Existing and proposed typical sections for main lanes, ramps, frontage roads, ancillary roads, and cross roads, including the following elements:
  - Proposed and future lane and shoulder widths
  - Existing and proposed roadway centerline
  - Direction of travel and turn movements allowed (right/left turn, option lane, etc.)
  - Curb offsets
  - Median widths
  - Clear zone widths
  - Existing and proposed ROW width
  - Pedestrian and bicycle accommodations
  - Sidewalk and bicycle accommodation widths and cross slopes, including buffer widths
  - Slope rates of front and back slopes
  - Roadside barrier nominal widths and offsets
- If the proposed design is interim, include typical sections for the ultimate design with the above parameters indicated
- Locations of wildlife crossing structures
- Vertical location of cross-drainage structures in the profile view
- Rivers, major stream crossings, and FEMA floodplain areas reflect the anticipated water surface for the required design event
- Locations and composition of existing and proposed freeway signing
- A written explanation or prepared exhibits of the sequence and methods of staged construction, including the treatment of crossovers and ramps

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

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

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

All schematics are to be submitted to the Design Division for review. The Design Division will provide approvals for schematics on the NHS and Interstate, as well as Major Projects and any other project identified on the FHWA's Projects of Division Interest (PoDI) list. If a project is on the PoDI and the project's PoDI Risk Analysis Stewardship & Oversight Plan (PoDI Plan) identifies that the project's schematic is to be reviewed by the FHWA, the Design Division will submit the schematic to the FHWA for review. For all other schematics, approvals will be at the District.
Section 4 — Access to the Interstate System

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

◆ Operational and Safety
◆ Access and Design

According to the 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 regardless of the funding for the new access points. This applicability includes routes incorporated into the interstate system under the provisions of 23 U.S.C. 103(c)(4)(A) or other legislation.

◆ Routes approved as a future part of the interstate system under 23 U.S.C. 103(c)(4)(B) represent a special case because they are not yet a part of the interstate system. Any proposed new or significant changes in access beyond those covered in the agreement, regardless of funding must be approved by FHWA.

◆ Toll roads that are incorporated into the interstate system do not require FHWA coordination and approval, except for segments where federal funds have been expended or these funds will be used for roadway improvements, or where the toll road section has been added to the interstate system under the provisions of 23 U.S.C. 103(c)(4)(A). The term “segment” is defined as the project limits described in the Federal-aid project agreement.

◆ Each break in the control of access to the interstate system right-of-way is considered to be an access point. Each entrance or exit point, including “locked gate” access, is considered to be an access point. For example, a diamond interchange configuration has four access points.

◆ Ramps providing access to rest areas, information centers, and weigh stations within the interstate controlled access are not considered access points for the purpose of 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 shall be reevaluated if the project has not progressed to construction within 3 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 to the May 2017 FHWA two-point policy. Refer to the TxDOT Policy for Interstate Access Justification Reports (IAJRs) and Project Development Process Manual for further guidance on the development of Interstate Access Justification Reports and submittal process through the Design Division. A copy of current TxDOT IAJR policy is available on Design Division webpage.
Section 5 — Preliminary Design

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

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

NOTE: Brackets [ ] indicate resultant documentation.

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

Surveying: See TxDOT Survey Manual
Design Exceptions, Waivers, Variances, and Deviations: See Roadway Design Manual (RDM) and Project Development Process Manual (PDP)
Schematics and IAJRs: See RDM and PDP Manuals
ROW establishment: See ROW Acquisition Manual
Driveways: See Access Management Manual
PS&E documentation: See PS&E Preparation Manual and PDP Manual
Utility documentation: See ROW Utilities Manual
Pavement design: See Pavement Manual
Bridge & bridge class structures: See Bridge Project Development Manual
Retaining wall layouts: See Geotechnical Manual
Hydraulic analysis: See Hydraulic Design Manual
Speed Zones: See Procedures for Establishing Speed Zones Manual
Illumination: See Highway Illumination Manual
Landscape and Aesthetic Design: See Landscape and Aesthetics Design Manual
Design Build Projects: See Design-Build Contract Administration Manual
NOTE: Work on or within 500-ft of railroad right of way, as measured longitudinally along the roadway, should be evaluated for impacts. Railroad coordination and letter is required for any work on or within 50-ft of railroad right of way.
Section 6 — Maintenance Considerations in Design

Maintenance

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

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

- Acquire drainage easements when necessary to grade outfalls and thus provide adequate drainage. Avoid instances where adjacent property elevation is well above the drainage outfall as this may form a dam at the outfall to the structure.

- Where practical, try to match the drainage structure to the natural grade of the drainage channel, and then profile the roadway over the structure. This practice may reduce siltation in the structure and erosion at the outfall.

- Avoid placing signs in the ditch. Such placement may impede drainage (making mowing more difficult) and result in erosion or siltation around the sign support.

- Where practical, riprap mow strips around sign supports may minimize the need for herbicidal treatment.

- At exit gores, try to extend the riprap area to include any EXIT sign supports. Extending the riprap will eliminate the need to mow or hand trim around the sign supports and keep mowers further from traffic.

- Address access control discrepancies (perhaps due to changes in property ownership) at ramp gores during design.

- Avoid the use of roadside barriers if the fixed object (culvert, large sign, steep slope, etc.) can be appropriately relocated or eliminated. The barrier itself represents a fixed object and should only be used where alternatives are impractical.

- When designing grade separations, consider extending riprap on the header banks of the overpasses all the way to the cross road pavement. This eliminates the need to mow or maintain a small strip of soil under the structure.

- Consider the provision of a narrow mow strip at the bottom or top of retaining walls to simplify mowing operations along the wall. Riprap considerations may also be appropriate in other locations (sign structures, narrow borders, etc.).
• Generally, designs should reduce the amount of hand trimming that would be required and eliminate the places that are relatively difficult for mowers to access.

• Provide access to areas requiring maintenance (mowing, bridge inspection, etc.). For twin bridge structures, provide sufficient distance (typically 10 ft minimum between) to facilitate access for inspections that may require aerial-vehicles.

• To the extent practical, utilization of desirable design criteria recommended herein regarding maximum roadway side slope ratios and ditch profile grades will reduce maintenance and make required maintenance operation easier to accomplish.
Chapter 2 — Basic Design Criteria

Contents:

Section 1 — Functional Classifications
Section 2 — Traffic Characteristics
Section 3 — Sight Distance
Section 4 — Horizontal Alignment
Section 5 — Vertical Alignment
Section 6 — Cross Sectional Elements
Section 7 — Drainage Facility Placement
Section 8 — Roadways Intersecting Department Projects
Section 1 — Functional Classifications

Overview

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

- **Principal arterial** - main movement (high mobility, limited access)
- **Minor arterial** - interconnects principal arterials (moderate mobility, limited access)
- **Collectors** - connects local roads to arterials (moderate mobility, moderate access)
- **Local roads and streets** - permits access to abutting land (high access, limited mobility)
Section 2 — Traffic Characteristics

Overview

Information on traffic characteristics is vital in selecting the appropriate geometric features of a roadway. Necessary traffic data includes traffic volume, traffic speed, and percentage of trucks or other large vehicles.

Traffic Volume

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

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

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

For two-lane rural highways, the DHV is the total traffic in both directions of travel. On highways with more than two lanes (or on two-lane roads where important intersections are encountered or where additional lanes are to be provided later), knowledge of the directional distribution of traffic during the design hour (DDHV) is essential for design. DHV and DDHV may be determined by the application of conversion factors to ADT.

Computation of DHV and DDHV. The percent of ADT occurring in the design hour (K) may be used to convert ADT to DHV as follows:

\[ DHV = (ADT)(K) \]

The percentage of the design hourly volume that is in the predominant direction of travel (D) and K are both considered in converting ADT to DDHV as shown in the following equation:

\[ DDHV = (ADT)(K)(D) \]
**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, D factors of 60 to 70 percent frequently occur.

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

**Projected Traffic Volumes.** Projected traffic volumes are provided by the Transportation Planning and Programming (TPP) Division upon request and serve as a basis for design of proposed improvements. For high-volume facilities, a tabulation showing traffic converted to DHV or DDHV will be provided by TPP if specifically requested. Generally, however, projected traffic volume is expressed as ADT with K and D factors provided.

**NOTE:** If the directional ADT is known for only one direction, total ADT may be computed by multiplying the directional ADT by two for most cases.

**Service Flow Rate.** A facility should be designed to provide sufficient capacity to accommodate the design traffic volumes (ADT, DHV, DDHV). The necessary capacity of a roadway is initially based on a set of “ideal conditions.” These conditions are then adjusted for the “actual conditions” that are predicted to exist on the roadway section. This adjusted capacity is termed 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
- Distribution of vehicle type.

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

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

All roadway design should reflect proper consideration of capacity and level of service procedures as detailed in the TRB’s *Highway Capacity Manual.*
Speed

Speed is one of the most important factors considered by travelers in selecting alternative routes or transportation modes. The speed of vehicles on a road depends, in addition to capabilities of the drivers and their vehicles, 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 either 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 these 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 manner with efficient traffic operations and with low crash frequency and severity. The facility should, therefore, accommodate nearly all demands with reasonable adequacy and also should only fail under severe or extreme traffic demands. Because only a small percentage of drivers travel at extremely high speed, it is not economically practical to design for them. They can use the roadway, of course, but will be constrained to travel at speeds less than they consider desirable. On the other hand, the speed chosen for design should not be that used by drivers 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- and high-speed designs. For design purposes, the following definitions apply:

- **Low-speed** is 45 mph [70 km/h] and below, and
- **High-speed** is 50 mph [80 km/h] and above.

Several tables and figures for high-speed conditions will show values for 45 mph [70 km/h] to provide information for transitional roadway sections.

**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 a logical one with respect to the anticipated operating speed, topography, the adjacent land use, modal mix, and the functional classification of the roadway. In selec-
tion of design speed, every effort should be made to attain a desired combination of safety, mobility, and efficiency within the constraints of environmental quality, economics, aesthetics, and social or political impacts. The selected design speed should be consistent with the speeds that drivers are likely to travel on a given roadway. A roadway of higher functional classification may justify a higher design speed than a lesser classified facility in similar topography. A low design speed, however, should not be selected where the topography is such that drivers are likely to travel at high speeds.

Selection of design speed for a given functionally classified roadway is influenced primarily by the character of terrain, economic considerations, extent of roadside development (i.e., urban or rural), and highway type. For example, the design speed chosen would usually be less for rough terrain, or for an urban facility with frequent points of access, as opposed to a rural highway on level terrain. Choice should be influenced by the expectations of drivers, which are closely related to traffic volume conditions, potential traffic conflicts, and topographic features.

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

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

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

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

The average running speed on a given roadway varies during the day, depending primarily on the traffic volume. Therefore, when reference is made to a running speed, it should be clearly stated whether this speed represents peak hours, off-peak hours, or an average for the day. Peak and off-peak running speeds are used in design and operation; average running speeds for an entire day are used in economic analyses. The effect of traffic volume on average running speed can be determined using the procedures of the Highway Capacity Manual.

Terrain

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

Turning Roadways and Intersection Corner Radii

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

Safety

TxDOT is developing additional strategies to incorporate safety into its system, contributing to the goal of eliminating traffic deaths statewide by 2050 (Vision Zero). Towards this end, the department uses a number of initiatives related to developing and operating a safer highway system.

Highway Safety Improvement Program (HSIP). HSIP is a federally funded program administered by TxDOT's Traffic Safety Division that allows highway safety improvements through strategic safety planning and performance measures. The HSIP requires states to develop and implement a Strategic Highway Safety Plan (SHSP). The purpose of the SHSP is to identify and analyze highway safety problems and correction opportunities, include projects or strategies to address them, evaluate the accuracy of data, and prioritize the proposed improvements. The SHSP
establishes five target areas including number and rate of fatalities, serious injuries, and non-motorized fatalities & serious injuries.

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

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

\[
\text{Crash Rate} = \frac{100,000,000 \times A}{365 \times T \times V \times L}
\]

Where:
- \(A\) = Number of reported crashes (in section or at location)
- \(T\) = Time frame of the analysis, years
- \(V\) = AADT, vehicles/day
- \(L\) = Length of section, miles

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

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

The Design Division has started a new section "Traffic Simulation and Safety Analysis". The purpose of the new section is to provide guidance and support for safety analysis. The Design Division is also developing a 'System Safety' tool which can be used to estimate a safety score of a particular roadway segment by selecting various design elements. The Rural Highways Tools (Two-Lane and Multi-Lane Rural) are available for use, with development currently underway for future Intersections, and Urban Highways Tools. Use of the applicable tools should begin during project scoping to evaluate the safety impacts of design decisions, as well as applicable projects in-progress to be let April 2020 and beyond. Please refer to the Design Division webpage for additional information and guidance on the tools.

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

Statewide average crash rates are used in the crash rate analysis method and are useful to compare against the crash rates of a particular highway segment/intersection. TxDOT maintains ten years of crash data. Summary reports of crash data are available at the following link: https://www.txdot.gov/government/enforcement/annual-summary.html

TxDOT uses “crashes per year”, level of crash severity, and crash type. Severity is defined on the KABCO scale as follows:

- K – Fatal Injury
- A – Incapacitating Injury
- B – Non-incapacitating injury
- C – Possible injury
- O – No Injury

Where accident frequencies include a wildlife-vehicle collision as a contributing factor in the CRIS records, consult with the District Environmental Coordinator or with Environmental Affairs Division to determine if a wildlife crossing structure could improve safety at these hot spot areas. The ENV Natural Resource Management Section and District Environmental Coordinators can provide information to conduct hot spot analysis and details on types of crossings, including schematics used within TxDOT and other states.
Section 3 — Sight Distance

Overview

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

- Stopping Sight Distance,
- Decision Sight Distance,
- Passing Sight Distance, and
- Intersection Sight Distance.

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 to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater lengths of visible roadway are desirable, the sight distance at every point along a roadway should be at least that needed for a below-average driver or vehicle to stop.

Stopping sight distance is the sum of two distances: (1) the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied; and (2) the distance needed to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively.

In computing and measuring stopping sight distances, the height of the driver’s eye is estimated to be 3.5 ft [1080 mm] and the height of the object to be seen by the driver is 2.0 ft [600 mm], equivalent to the taillight height of the passenger car.

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

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

Table 2-1: Stopping Sight Distance

<table>
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<th>Design Speed (mph)</th>
<th>Brake reaction distance (ft)</th>
<th>Braking distance on level (ft)</th>
<th>Calculated (ft)</th>
<th>Design (ft)</th>
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<td>275.6</td>
<td>539.9</td>
<td>815.5</td>
<td>820</td>
</tr>
<tr>
<td>80</td>
<td>294.0</td>
<td>614.3</td>
<td>908.3</td>
<td>910</td>
</tr>
</tbody>
</table>

Note: brake reaction distance predicated on a time of 2.5s; deceleration rate 11.2 ft/sec²

**Decision Sight Distance**

Decision sight distance is the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source, recognize the source, select an appropriate speed and path, and initiate and complete the required maneuver safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to maneuver their vehicles at the same or reduced speed rather than to just stop, its values are substantially greater than stopping sight distance. Table 2-2 shows recommended decision sight distance values for various avoidance maneuvers.
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-hand exits)
- Changes in cross section such as toll plazas and lane drops, and
- Areas of concentrated demand where there is apt to be “visual noise” whenever sources of information compete, as those from roadway elements, traffic, 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. During the design process, the roadway engineer can avoid the location of intersections within a reduced decision zone either by relocating the intersection or by changing the grades to reduce the size of the reduced design zone.
Passing Sight Distance

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

Intersection Sight Distance

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

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

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

- Intersections with no control
- Intersections with stop control on the minor road
- Intersections with yield control on the minor road
- Intersections with traffic signal control
- Intersections with all-way stop control
- Left turns from the major road.
Sight Distance at Under-Crossing

Sight distance through a grade crossing needs to be as long as the minimum stopping sight distance and preferably longer. Line of sight may cut by the structure and limit the sight distance to less than otherwise is attainable. Where practical, provide the minimum length of sag vertical curve at grade separated structures. See Figure 2-1. Sight Distance at Under-crossings.

Sag curves at under-crossings should be designed to provide vertical clearance for the largest legal vehicle that could use the under-crossing without a permit. For example, a WB-67 tractor-trailer will need a longer sag curve than a single-unit truck to avoid striking the overhead structure.

Figure 2-1. Sight Distance at Under-crossings. AASHTO 2018.

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

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

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

Where:
- $L$ = length of vertical curve, ft
- $S$ = sight distance, ft
- $C$ = vertical clearance, ft
- $h_1$ = eye height, ft
- $h_2$ = height of object, ft
- $A$ = algebraic difference in grades, percent
Case 2 - Sight distance less than length of vertical curve (S<L)

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

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

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

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

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

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

Where:

- \(L\) = length of vertical curve, ft
- \(A\) = algebraic difference in grades, percent
- \(S\) = sight distance, ft
- \(C\) = vertical clearance, ft
- \(h_1\) = height of eye, ft
- \(h_2\) = height of object, ft
Section 4 — Horizontal Alignment

Overview

In the design of highway alignment, it is necessary to establish the proper relation between design speed and curvature. The two basic elements of horizontal curves are **Curve Radius** and **Superelevation Rate**.

General Considerations for Horizontal Alignment

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

- Flatter than minimum curvature for a certain design speed should be used where possible, retaining the minimum guidelines for the most critical conditions.

- Compound curves should be used with caution and should be avoided on mainlanes where conditions permit the use of flat simple curves. Where compound curves are used, the radius of the flatter curve should not be more than 50 percent greater than the radius of the sharper curve for rural and urban open highway conditions. For intersections or other turning roadways (such as loops, connections, and ramps), this percentage may be increased to 100 percent.

- Alignment consistency should be sought. Sharp curves should not follow tangents or a series of flat curves. Sharp curves should be avoided on high, long fill areas.

- Reverse curves on high-speed facilities should include an intervening tangent section of sufficient length to provide adequate superelevation transition between the curves.

- Broken-back curves (two curves in the same direction connected with a short tangent) should normally not be used. This type of curve is unexpected by drivers and is not pleasing in appearance.

- Horizontal alignment and its associated design speed should be consistent with other design features and topography. Coordination with vertical alignment is discussed in **Combination of Vertical and Horizontal Alignment** in Section 5, Vertical Alignment.
Curve Radius

The minimum radii of curves are important control values in designing for safe operation. Design guidance for high speed or non-urban curvature is shown in Table 2-3 and Table 2-4; with and without superelevation respectively.

**Table 2-3: Horizontal Curvature of High-Speed and Non-Urban Highways with Superelevation**

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Usual Min.(^{1,2}) Radius of Curve (ft)</th>
<th>Absolute Min.(^{1,3}) Radius of Curve (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[based on emax = 6%]</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>810</td>
<td>643</td>
</tr>
<tr>
<td>50</td>
<td>1050</td>
<td>833</td>
</tr>
<tr>
<td>55</td>
<td>1635</td>
<td>1060</td>
</tr>
<tr>
<td>60</td>
<td>2195</td>
<td>1330</td>
</tr>
<tr>
<td>65</td>
<td>2740</td>
<td>1660</td>
</tr>
<tr>
<td>70</td>
<td>3390</td>
<td>2040</td>
</tr>
<tr>
<td>75</td>
<td>3750</td>
<td>2500</td>
</tr>
<tr>
<td>80</td>
<td>4575</td>
<td>3050</td>
</tr>
<tr>
<td></td>
<td>[based on emax = 8%]</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>740</td>
<td>587</td>
</tr>
<tr>
<td>50</td>
<td>955</td>
<td>758</td>
</tr>
<tr>
<td>55</td>
<td>1480</td>
<td>960</td>
</tr>
<tr>
<td>60</td>
<td>1980</td>
<td>1200</td>
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<td>65</td>
<td>2445</td>
<td>1480</td>
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<td>3315</td>
<td>2210</td>
</tr>
<tr>
<td>80</td>
<td>4005</td>
<td>2670</td>
</tr>
</tbody>
</table>

\(^{1}\) For other maximum superelevation rates refer to AASHTO’s *A Policy on Geometric Design of Highways and Streets*.

\(^{2}\) Applies to new location construction. For 3R or reconstruction, existing curvature equal to or flatter than absolute minimum values may be retained unless crash history indicates flattening curvature.

\(^{3}\) Absolute minimum values should be used only where unusual design circumstances dictate.
Chapter 2 — Basic Design Criteria  
Section 4 — Horizontal Alignment

For high speed design conditions, the maximum deflection angle allowable without a horizontal curve is fifteen (15) minutes. For low speed design conditions, the maximum deflection angle allowable without a horizontal curve is thirty (30) minutes.

**Superelevation Rate**

As a vehicle traverses a horizontal curve, centrifugal force is counter-balanced by the vehicle weight component due to roadway superelevation and by the side friction between tires and surfacing as shown in the following equation:

\[ e + f = \frac{V^2}{15R} \quad \text{(US Customary)} \]

*Where:*
- \( e \) = superelevation rate, in decimal format
- \( f \) = side friction factor
- \( V \) = vehicle speed, mph
- \( R \) = curve radius, feet

---

### Table 2-4: Horizontal Curvature of High-Speed and Non-Urban Highways without Superelevation (2% Normal Crown Maintained)

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>6% Min. Radius (ft)</th>
<th>8% Min. Radius (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>868</td>
<td>932</td>
</tr>
<tr>
<td>20</td>
<td>1580</td>
<td>1640</td>
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<tr>
<td>25</td>
<td>2290</td>
<td>2370</td>
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<tr>
<td>30</td>
<td>3130</td>
<td>3240</td>
</tr>
<tr>
<td>35</td>
<td>4100</td>
<td>4260</td>
</tr>
<tr>
<td>40</td>
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<td>5410</td>
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<tr>
<td>45</td>
<td>6480</td>
<td>6710</td>
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<tr>
<td>50</td>
<td>7870</td>
<td>8150</td>
</tr>
<tr>
<td>55</td>
<td>9410</td>
<td>9720</td>
</tr>
<tr>
<td>60</td>
<td>11100</td>
<td>11500</td>
</tr>
<tr>
<td>65</td>
<td>12600</td>
<td>12900</td>
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<tr>
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<td>14100</td>
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<tr>
<td>75</td>
<td>15700</td>
<td>16100</td>
</tr>
<tr>
<td>80</td>
<td>17400</td>
<td>17800</td>
</tr>
</tbody>
</table>

For high speed design conditions, the maximum deflection angle allowable without a horizontal curve is fifteen (15) minutes. For low speed design conditions, the maximum deflection angle allowable without a horizontal curve is thirty (30) minutes.
There are practical limits to the rate of superelevation. High rates create steering problems for drivers traveling at lower speeds, particularly during ice or snow conditions. On urban facilities, lower maximum superelevation rates may be employed since adjacent buildings, lower design speeds, and frequent intersections are limiting factors.

Although maximum superelevation is not commonly used on urban streets, if provided, maximum superelevation rates of 4 percent should be used. For urban freeways and all types of rural highways, maximum rates of 6 to 8 percent are generally used.

Superelevation on Low-Speed Facilities. Although superelevation is advantageous for traffic operations, various factors often combine to make its use impractical in many built-up 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 streets in urban areas are frequently designed without superelevation, and centrifugal force is counteracted solely with side friction.

Table 2-5 shows the relationship of radius, superelevation rate, and design speed for low-speed urban street design. For example, for a curve with normal crown (2 percent cross slope each direction), the designer may enter Table 2-5 with a given curve radius of 400 ft [110 m] and determine that through interpolation, the related design speed is approximately:

- 35 mph for positive crown condition, and
- 32 mph for negative crown condition.

Table 2-5 should be used to evaluate existing conditions and may be used in design for constrained conditions, such as detours.

When superelevation is used on low-speed streets, Table 2-5 should be used to determine design superelevation rate for specific curvature and design speed conditions. Given a design speed of 35 mph and a 400 ft radius curve, Table 2-5 indicates an approximate superelevation rate of 2.4 percent.
### Table 2-5: Minimum Radii and Superelevation for Low-Speed Urban Streets

<table>
<thead>
<tr>
<th>e(%)</th>
<th>V = 15 mph R (ft)</th>
<th>V = 20 mph R (ft)</th>
<th>V = 25 mph R (ft)</th>
<th>V = 30 mph R (ft)</th>
<th>V = 35 mph R (ft)</th>
<th>V = 40 mph R (ft)</th>
<th>V = 45 mph R (ft)</th>
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<tr>
<td>-4.0</td>
<td>54</td>
<td>116</td>
<td>219</td>
<td>375</td>
<td>583</td>
<td>889</td>
<td>1227</td>
</tr>
<tr>
<td>-3.0</td>
<td>52</td>
<td>111</td>
<td>208</td>
<td>353</td>
<td>544</td>
<td>821</td>
<td>1125</td>
</tr>
<tr>
<td>-2.8</td>
<td>51</td>
<td>110</td>
<td>206</td>
<td>349</td>
<td>537</td>
<td>808</td>
<td>1107</td>
</tr>
<tr>
<td>-2.6</td>
<td>51</td>
<td>109</td>
<td>204</td>
<td>345</td>
<td>530</td>
<td>796</td>
<td>1089</td>
</tr>
<tr>
<td>-2.4</td>
<td>51</td>
<td>108</td>
<td>202</td>
<td>341</td>
<td>524</td>
<td>784</td>
<td>1071</td>
</tr>
<tr>
<td>-2.2</td>
<td>50</td>
<td>108</td>
<td>200</td>
<td>337</td>
<td>517</td>
<td>773</td>
<td>1055</td>
</tr>
<tr>
<td>-2.0</td>
<td>50</td>
<td>107</td>
<td>198</td>
<td>333</td>
<td>510</td>
<td>762</td>
<td>1039</td>
</tr>
<tr>
<td>-1.5</td>
<td>49</td>
<td>105</td>
<td>194</td>
<td>324</td>
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<td>47</td>
<td>99</td>
<td>181</td>
<td>300</td>
<td>454</td>
<td>667</td>
<td>900</td>
</tr>
<tr>
<td>1.5</td>
<td>45</td>
<td>94</td>
<td>170</td>
<td>279</td>
<td>419</td>
<td>610</td>
<td>818</td>
</tr>
<tr>
<td>2.0</td>
<td>44</td>
<td>92</td>
<td>167</td>
<td>273</td>
<td>408</td>
<td>593</td>
<td>794</td>
</tr>
<tr>
<td>2.2</td>
<td>44</td>
<td>91</td>
<td>165</td>
<td>270</td>
<td>404</td>
<td>586</td>
<td>785</td>
</tr>
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<td>2.4</td>
<td>44</td>
<td>91</td>
<td>164</td>
<td>268</td>
<td>400</td>
<td>580</td>
<td>776</td>
</tr>
<tr>
<td>2.6</td>
<td>43</td>
<td>90</td>
<td>163</td>
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<td>89</td>
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<td>263</td>
<td>393</td>
<td>567</td>
<td>758</td>
</tr>
<tr>
<td>3.0</td>
<td>43</td>
<td>89</td>
<td>160</td>
<td>261</td>
<td>389</td>
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<td>750</td>
</tr>
<tr>
<td>3.2</td>
<td>43</td>
<td>88</td>
<td>159</td>
<td>259</td>
<td>385</td>
<td>556</td>
<td>742</td>
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<td>3.4</td>
<td>42</td>
<td>88</td>
<td>158</td>
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<td>382</td>
<td>550</td>
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<td>42</td>
<td>87</td>
<td>157</td>
<td>254</td>
<td>378</td>
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<td>42</td>
<td>87</td>
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<td>252</td>
<td>375</td>
<td>539</td>
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<td>42</td>
<td>86</td>
<td>154</td>
<td>250</td>
<td>371</td>
<td>533</td>
<td>711</td>
</tr>
</tbody>
</table>

**Notes:**
2. Superelevation may be optional on low-speed urban streets.
3. Negative superelevation values beyond -2.0% should be used for low type surfaces such as gravel, crushed stone, and earth. However, areas with intense rainfall may use normal cross slopes on high type surfaces of -2.5%.
**Superelevation Rate on High-Speed Facilities.** Tables 2-6 and 2-7 show superelevation rates (maximum 6 and 8 percent, respectively) for various design speeds and radii. These tables should be used for high-speed facilities such as rural highways and urban freeways.

Table 2-6: High Speed and Non-Urban Minimum Radii for Design Superelevation Rates, Design Speeds, and $\text{emax} = 6\%$

<table>
<thead>
<tr>
<th>$e$ (%)</th>
<th>$V_d = 15$ mph R (ft)</th>
<th>$V_d = 20$ mph R (ft)</th>
<th>$V_d = 25$ mph R (ft)</th>
<th>$V_d = 30$ mph R (ft)</th>
<th>$V_d = 35$ mph R (ft)</th>
<th>$V_d = 40$ mph R (ft)</th>
<th>$V_d = 45$ mph R (ft)</th>
<th>$V_d = 50$ mph R (ft)</th>
<th>$V_d = 55$ mph R (ft)</th>
<th>$V_d = 60$ mph R (ft)</th>
<th>$V_d = 65$ mph R (ft)</th>
<th>$V_d = 70$ mph R (ft)</th>
<th>$V_d = 75$ mph R (ft)</th>
<th>$V_d = 80$ mph R (ft)</th>
</tr>
</thead>
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<td>1120 1630 2240 2950 3770 4680 5700 6820 8060 9130 10300 11500 12900</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.2</td>
<td>543 991 1450 2000 2630 3370 4190 5100 6110 7230 8200 9240 10400 11600</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.4</td>
<td>482 884 1300 1790 2360 3030 3770 4600 5520 6540 7430 8380 9420 10600</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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</tr>
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<td></td>
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<td></td>
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<td></td>
</tr>
<tr>
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<tr>
<td>3.2</td>
<td>300 566 850 1200 1600 2080 2600 3200 3860 4600 5280 6010 6810 7680</td>
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</tr>
<tr>
<td>3.4</td>
<td>256 498 761 1080 1460 1900 2390 2940 3560 4250 4890 5580 6340 7180</td>
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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 be effected over a length adequate for the usual travel speeds.

Desirable design values for length of superelevation transition are based on using a given maximum relative gradient between profiles of the edge of traveled way and the axis of rotation. Table 2-8 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.
Transition length, L, for a multilane highway can be calculated using the following equation:

\[ L_{CT} = \frac{((CS)(W))}{G} \]  

**US Customary**

Where:

- \( L_{CT} \) = calculated transition length (ft)
- \( CS \) = percent change in cross slope of superelevated pavement,
- \( W \) = distance between the axis of rotation and the edge of traveled way (ft),
- \( G \) = maximum relative gradient (Table 2-8: Maximum Relative Gradient for Superelevation Transition).

Example determinations of superelevation transition shown in Figure 2-2.
As the number of lanes to be transitioned increases, the length of superelevation transition increases proportionately with the increased width. While strict adherence to the length ($L_{CT}$) calculation is desirable, the length for multilane highways may become impractical for design purposes (e.g. drainage problems, avoiding bridges, accommodating merge/diverge condition). A minimum length $L_{CT}$, can be calculated using adjustment factors as shown in Table 2-9, such that the transition length formula becomes:

$$L_{CT} = b[(CS)(W)]/G \text{ (US Customary and Metric)}$$

where “b” is defined in Table 2-9.
Superelevation Transition Placement

The location of the transition in respect to the termini of a simple (circular) curve should be placed to minimize lateral acceleration and the vehicle's lateral motion. The appropriate allocation of superelevation transition on the tangent, either preceding or following a curve, is provided on Table 2-10. When spiral curves are used, the transition usually is distributed over the length of the spiral curve.

Table 2-10: Portion of Superelevation Transition Located on the Tangent

<table>
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<th>Design Speed (mph)</th>
<th>No. of Lanes Rotated</th>
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<tr>
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<td>0.80</td>
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<tr>
<td>50 - 80</td>
<td>0.70</td>
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</table>

1 These values are desirable and should be followed as closely as possible when conditions allow. A value between 0.6 and 0.9 for all speeds and rotated widths is considered acceptable. (AASHTO's A Policy on Geometric Design of Highways and Streets).

Care must be exercised in designing the length and location of the transition. Profiles of both gutters or pavement edges should be plotted relative to the profile grade line to insure proper drainage, especially where these sections occur within vertical curvature of the profile grade line. Special care should be given to ensure that the zero cross slope in the superelevation transition does not occur near the flat portion of the crest or sag vertical curve. A plot of roadway contours can identify drainage problems in areas of superelevation transition. See "Minimum Transition Grades" section of AASHTO's A Policy on Geometric Design of Highways and Streets for further discussion on potential drainage problems and effective means to mitigate them.
Whenever reverse curves are closely spaced and superelevation transition lengths overlap, L values should be adjusted to prorate change in cross slope and to ensure that roadway cross slopes are in the proper direction for each horizontal curve.

**Superelevation Transition Type**

Linear superelevation transition may be used. Where appearance is a factor (e.g. curbed sections and retaining walls) use of reverse parabolas is recommended for attaining superelevation as shown in Figure 2-2. This produces an outer edge profile that is smooth, undistorted, and pleasing in appearance. Sufficient information needs to be in the plans to ensure the parabolic design is properly constructed.

Figure 2-2 shows reverse parabolas over the full length of the transition. Alternative methods for developing smooth-edge profiles over the length of the transition are given in the section "Design of Smooth Profiles for Traveled Way Edges" of AASHTO's *A Policy on Geometric Design of Highways and Streets*.

**Sight Distance on Horizontal Curves**

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

The following equation applies only to the circular curves longer than the stopping sight distance for the pertinent design speed. For example, with a 50 mph [80 km/h] design speed and a curve with a 1150 ft [350 m] radius, a clear sight area with a middle ordinate of a approximately 20 ft [6.0 m] is needed for stopping sight distance.

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

Where:
- \( M \) = middle ordinate (ft)
- \( S \) = stopping sight distance (ft) and,
- \( R \) = radius (ft)

Figure 2-3 provides a graph illustrating the required offset where stopping sight distance is less than the length of curve (\( S < L \)).
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, high ground, etc., do not restrict sight below that required in either direction.

Where sufficient stopping sight distance is not available because a railing or a longitudinal barrier constitutes a sight obstruction, alternative designs should be considered. The alternatives are:

1. 

**STOPPING SIGHT DISTANCE ON HORIZONTAL CURVES**

   *(US CUSTOMARY)*

   **Figure 2-3. (US). Stopping Sight Distance on Horizontal Curves.**
increase the offset to the obstruction, (2) increase the radius, or (3) reduce the design speed. However, the alternative selected should not incorporate shoulder widths on the inside of the curve in excess of 12 ft [3.6 m] because of the concern that drivers will use wider shoulders as a passing or travel lane.
Section 5 — Vertical Alignment

Overview

The two basic elements of vertical alignment are Grades and Vertical Curves.

Grades

The effects of rate and length of grade are more pronounced on the operating characteristics of trucks than on passenger cars and thus may introduce undesirable speed differentials between the vehicle types. The term “critical length of grade” is used to indicate the maximum length of a specified ascending gradient upon which a loaded truck can operate without an unreasonable reduction in speed (commonly 10 mph [15 km/h]). Figure 2-4 shows the relationship of percent upgrade, length of grade, and truck speed reduction for an entering speed of 70 mph. Where critical length of grade is exceeded for two-lane highways, climbing lanes should be considered as discussed in the TRB’s Highway Capacity Manual.

![Figure 2-4. Critical Lengths of Grade for Design (70mph Entering Speed).](image-url)
Table 2-11 summarizes the maximum grade controls in terms of design speed. Generally, maximum design grade should be used infrequently rather than as a value to be used in most cases. However, for certain cases such as urban freeways, a maximum value may be applied in blanket fashion on interchange and grade separated approaches.

**Table 2-11: Maximum Grades**

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¹ 8% maximum in commercial areas on local streets, desirably less than 5%. Flatter grades should be used where practical.

Flat or level grades on uncurbed pavements are satisfactory when the pavement is adequately crowned to drain the surface water laterally. When side ditches are required, the roadway grade should seldom be less than 0.5 percent for unpaved ditches and 0.25 percent for lined channels. With curbed pavements, desirable minimum grades of 0.35 percent should be provided to facilitate surface drainage. Joint analyses of rainfall frequency and duration, the longitudinal grade, cross sections, and the highway volume must be made.
slope, curb inlet type and spacing of inlets or discharge points usually is required so that the width of water on the pavement surface during likely storms does not unduly interfere with traffic. Criteria for water ponding for various functionally classified roadways are contained in the *Hydraulic Design Manual*.

### Vertical Curves

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

![Vertical Curve Diagram](image)

**US CUSTOMARY EQUATION**

1. \[ E = \frac{A}{800} \]
2. \[ y = \frac{p^2 E}{L^2} \]
   \[ y = \frac{p^2 A}{200L} \]

**Where:**

- \( G_1, G_2 \) = Tangent grades, in percent
- \( E \) = Ordinate from P.I. to curve, in feet
- \( A \) = \( G_1 - G_2 \), The algebraic difference in grade
- \( L \) = Length of curve, in feet
- \( y \) = Ordinate from tangent to curve, in feet
- \( D \) = Distance from nearest P.C. or P.I. to any point on curve, in feet

**VERTICAL CURVE (US CUSTOMARY)**

*Figure 2-5. Vertical Curve.*

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

\[ L = \text{length of vertical curve}; \]
S = sight distance for crest vertical curves or headlight beam distance for sag vertical curves;
A = algebraic difference in grades, percent;
K = length of vertical curve per percent change in A (also known as the design control)

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 Figures 2-6 and 2-7. The solid lines give the minimum vertical curve lengths on the basis of rounded values of K. These lengths represent minimum values based on design speed and longer curves are desired wherever practical.

A dashed curve crossing the solid lines indicates where 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 [70 km/h]. Also, for small values of A, the vertical curve lengths are zero because the sight line passes over the apex. Since this relationship does not represent desirable design practice except in limited conditions (see discussion on Grade Change Without Vertical Curves), a minimum length of vertical curve is shown. Attention should be given where there are successive vertical curves.

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

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

The minimum length of vertical curves (both crest and sag) are expressed as approximately three times the design speed in miles per hour \(L_{\text{min}} = 3V\) or 0.6 times the design speed in kilometers per hour \(L_{\text{min}} = 0.6V\). However, these minimum lengths are not considered a design control (i.e., a design exception would not be required for these minimum length values as long as the minimum K value for the relevant design speed is met).
There is a level point on a vertical curve which can affect drainage; particularly on curbed facilities. Typically, there is no difficulty with drainage on highways if the curve is sharp enough so that a minimum grade of 0.30 percent is reached at a point about 50 ft [15 m] from the crest or sag. This corresponds to a K value of 167 [51 m] which is plotted in Figures 2-6 and 2-7 as the drainage threshold. All combinations above or to the left of this line satisfy the drainage criterion. The combinations below and to the right of this line involve flatter vertical curves. Special attention is needed in these cases to ensure proper pavement drainage. It is not intended that these values be considered a design maximum, but merely a value beyond which drainage should be more carefully designed.

Figure 2-6. (US). Design Controls for Crest Vertical Curves.
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 about 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.
Care should be exercised in sag vertical curve design to insure that overhead sight obstructions such as structures for overpassing roadways, overhead sign bridges, tree crowns, etc., do not reduce stopping sight distance below the appropriate minimum value.

**Grade Change without Vertical Curves**

Designing a sag or crest vertical point of intersection without a vertical curve is generally acceptable where the grade difference (A) is:

- 1.0 percent or less for design speeds equal to or less than 45 mph [70 km/h], and
- 0.5 percent or less for design speeds greater than 45 mph [70 km/h].

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

- Bridges (including bridge ends),
- Direct-traffic culverts, and
- Other locations requiring carefully detailed grades.

**Combination of Vertical and Horizontal Alignment**

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

- The design speed of both vertical and horizontal alignment should be compatible with longer vertical curves and flatter horizontal curves than dictated by minimum values. Design speed should be compatible with topography with the roadway fitting the terrain where feasible.
- Alignment should be as flat as possible near intersections where sight distance is important.
- For rural divided facilities, independent mainlane profiles are often more aesthetic and economical. Where used on non-controlled access facilities with narrow medians, care should be exercised in the location of median openings to minimize crossover grades and insure adequate sight distance for vehicles stopped therein.
- When designing independent vertical and horizontal profiles on divided facilities, considerations should be given to the impact these profiles may have on future widening into the median.
- For two-lane rural highways and Super 2 Highways the need for safe passing sections at frequent intervals should be carefully considered in developing horizontal and vertical alignments.
Section 6 — Cross Sectional Elements

Overview

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

- Pavement Cross Slope
- Median Design
- Lane Widths
- Shoulder Widths
- Sidewalks and Pedestrian Elements
- Curb and Curb and Gutters
- Roadside Design
- Slopes and Ditches
- Lateral Offset to Obstructions
- Clear Zone

Pavement design is covered in TxDOT’s Pavement Design Guide.

Pavement Cross Slope

The operating characteristics of vehicles on crowned pavements are such that on 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 steep cross slope is desirable to contain the flow of water adjacent to the curb. The recommended pavement cross slope for usual conditions is 2 percent. In areas of high rainfall, steeper cross slopes may be used (see AASHTO’s A Policy on Geometric Design of Highways and Streets).

On multilane divided highways, pavements with three or more lanes inclined in the same direction desirably should have greater slope across the outside lane(s) than across the two interior lanes. The increase in slope in the outer lane(s) should be at least 0.5 percent greater than the inside lanes. In these cases, the inside lanes may be sloped flatter than normal, typically at 1.5 percent but not less than 1.0 percent. A cross slope should not normally exceed 3 percent on a tangent alignment unless there are three or more lanes in one direction of travel.
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 1.0 percent.

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

- Bituminous and concrete-surface shoulders should be sloped from 2 to 6 percent (often the slope rate is identical to that used on the travel lanes),
- Gravel or crushed rock shoulders should be sloped from 4 to 6 percent, and
- Turf shoulders should be sloped at about 8 percent.

Pavement cross slopes on all roadways, exclusive of superelevation transition sections, should not be less than 1 percent.

**Median Design**

A median (i.e., the area between opposing travel lane edges) is provided primarily to separate opposing traffic streams. The general range of median width is from 4 ft to 76 ft [1.2 m to 22.8 m], with design width dependent on the type and location of the highway or street facility.

In rural areas, median sections are normally wider than in urban areas. For multi-lane rural highways without access control, a median width of 76 ft [22.8 m] is desirable to provide complete shelter for trucks at median openings (crossovers). These wide, depressed medians are also effective in reducing headlight glare and providing a clear zone for run-off-the-road vehicle encroachments.

Where economically feasible, freeways in rural areas should also desirably include a 76 ft [22.8 m] median. Since freeways by design do not allow at-grade crossings, median widths need not be sufficient to shelter crossing trucks. In this regard, where right-of-way costs are prohibitive, reduced median widths (less than 76 ft [22.8 m]) may be appropriate for certain rural freeways. Statistical studies have shown that over 90 percent of median encroachments involve lateral distances traveled of 48 ft [14.4 m] or less. In this regard, depressed medians on rural freeways sections should be 48 ft [14.4 m] or more in width.
Urban freeways generally include narrower, flush medians with continuous longitudinal barriers. For urban freeways with flush median and six or more travel lanes, full (10 ft [3.0 m]) inside shoulders should be provided to provide space for emergency parking. Median widths vary up to 30 ft [9.0 m], with 24 ft [7.2 m] commonly used. For projects involving the rehabilitation and expansion of existing urban freeways, the provision of wide inside shoulders may not be feasible.

For low-speed urban arterial streets, flush or curbed medians are used. A width of 16 ft [4.8 m] will effectively accommodate left-turning traffic for either raised (turn lane plus raised median) or flush medians. However, where pedestrian refuge is a consideration for raised medians, allowances for a 6 ft width raised median from BOC to BOC is preferred for pedestrian refuge, see Figure 2-10 (6 ft from FOC to FOC is the minimum requirement). Where the need for dual left turns are anticipated at cross streets, the median width should be 28 ft [8.4 m]. The two-way (continuous) left-turn lane design is appropriate where there exists (or is expected to exist) a high frequency of mid-block left turns. Median types for urban arterials without access control are further discussed in Chapter 3, Section 2, “Urban Streets”.

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

High severity injuries and fatalities are a result of cross median crashes on high speed roadways. All median related incidents begin with median encroachment. Reducing median encroachment reduces cross median crashes and fixed object crashes in the median. Median encroachment countermeasures should be considered where appropriate. 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
  - Provide continuous median barrier
- Reduce likelihood of vehicle overturning:
  - Flatten median slopes
  - Provide U-shaped (rather than V-shaped) median cross section
Provide barrier to shield steep slopes

Improve design of geometric elements:
- Provide wider median slopes
- Minimize sharp curves with radii less than 3,000 ft
- Minimize steep grades of 4% or more

Improve design of mainline ramp terminals:
- Increase separation between on-ramps and off-ramps
- Minimize left-hand exits
- Improve design of merge and diverge areas by lengthening speed-change lanes
- Simplify design of weaving areas
- 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
- 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 (eg., 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
- Raise the state of preparedness for winter maintenance
Lane Widths

For high-speed facilities such as all freeways and most rural arterials, lane widths should be 12 ft [3.6 m] minimum. For low-speed urban streets, 11 ft or 12 ft [3.3 m or 3.6 m] lanes are generally used. Subsequent sections of this manual identify appropriate lane widths for the various classes of highway and street facilities.

Bicycle accommodations must be considered when a project is scoped, and are a requirement for urban streets. For urban streets, the typical required accommodations will be either a bicycle lane, shared lane, or shared use path. See Ch. 6, Section 4 for additional guidance and design criteria. For rural projects, the minimum shoulder requirements are specified in the Shoulder width portion of this Section, although the additional accommodations for bicyclists as specified in Ch. 6, Section 4 are recommended for anticipated bicycle corridors.

Shoulder Widths

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

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

Shoulder widths should accommodate bicyclists where a designated bicycle lane or shared use path is not provided. When 4 ft or narrower shoulders are typical for a roadway, urban or rural, provide, a 5 ft minimum clear space (measured from edge of travel lane to toe of bridge rail) on bridges being replaced or rehabilitated. On divided highways, this guidance only applies to the outside shoulder. Exceptions to the 5 ft clear space are permitted for off-system facilities with less than 400 ADT.

On urban collector and local streets, parking lanes may be provided instead of shoulders. On arterial streets, parking lanes decrease capacity and generally are discouraged.
Sidewalks and Pedestrian Elements

Walking is an important transportation mode that needs to be incorporated in transportation projects. Planning for pedestrian facilities should occur early and continuously throughout project development. Sidewalks provide distinct separation of pedestrians and vehicles, serving to increase pedestrian safety as well as to enhance vehicular capacity. Sidewalks should be included on a project located in an urban setting where:

- Construction is within existing right-of-way, and the scope of work involves pavement widening; OR
- Full reconstruction or new construction that requires new right-of-way.

In typical suburban development, there may initially be relatively few pedestrian trips because there are few closely located pedestrian destinations. However, as development occurs and pedestrian demand increases, it is always difficult and more costly to retrofit pedestrian facilities if they were not considered in the initial design. Early consideration of pedestrian facility design during the project development process may also greatly simplify compliance with accessibility requirements established by the Americans with Disabilities Act (ADA) Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right of Way (PROWAG) and the Texas Accessibility Standards (TAS). Meeting requirements of PROWAG will meet or exceed TAS requirements.

Specific design requirements to accommodate the needs of persons with disabilities are established by the Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way (PROWAG), the Texas Accessibility Standards (TAS), and related rulemaking. A request for a design variance for any deviations from TAS requirements must be submitted to the Texas Department of Licensing and Regulation (TDLR) for approval.

Sidewalk Location. For pedestrian comfort, especially adjacent to high speed traffic, it is desirable to provide a buffer space between the traveled way and the sidewalk as shown in Figure 2-8(A). For curb and gutter sections, a buffer space of 4 ft to 6 ft [1.2m to 1.8m] between the back of the curb and the sidewalk is desirable. Roadways in urban and suburban areas without curb and gutter require sidewalks, which should be placed between the ditch and the right of way line if practical. Note that pedestrian street crossings must be ADA compliant.

Sidewalk Width. Sidewalks should be wide enough to accommodate the volume and type of pedestrian traffic expected in the area. The minimum clear sidewalk width is 5 ft [1525 mm]. Where a sidewalk is placed immediately adjacent to the curb as shown in Figure 2-8(B), a sidewalk width of 6 ft [1830 mm] is recommended to allow additional space for street and highway hardware and allow for the proximity of moving traffic. Sidewalk widths of 8 ft [2440 mm] or more may be appropriate in commercial areas, along school routes, and other areas with concentrated pedestrian traffic.
Where necessary to cross a driveway while maintaining the maximum 2 percent cross slope, the sidewalk width may be reduced to 4 ft [1220 mm] (Figure 2-9). Also, if insufficient space is available to locate street fixtures (elements such as sign supports, signal poles, fire hydrants, manhole covers, and controller cabinets that are not intended for public use) outside the 5 ft [1525 mm] minimum clear width, the sidewalk width may be reduced to 4 ft [1220 mm] for short distances, except just after a pedestrian ramp where a 5 ft width is required for wheelchair maneuverability.

Walking Speeds. Air temperature, time of day, trip purpose, age, gender, ability, grade, and presence of ice and snow all affect pedestrian walking speeds. Typical pedestrian walking speeds range from approximately 3.0 to 4.0 ft/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 ft/s is used, with a walking speed of 3.0 ft/s used where older pedestrians are expected or near schools.

Street Crossings. Reducing pedestrian-vehicular conflicts is important for designers to consider as we emphasize people transportation and not vehicular travel. Intersections can present formidable barriers to pedestrian travel. Intersection designs which incorporate properly placed curb ramps, sidewalks, crosswalks, pedestrian signal heads and pedestrian refuge islands can make the environment more accommodating for pedestrians. Desirably, drainage inlets should be located on the upstream side of crosswalks and sidewalk ramps.

Where at-grade crossings occur on sections with moderate to steep grades, it is desirable to reduce the grade through the intersection. Profile changes are beneficial to vehicles making turns and the cross slope of the pedestrian crosswalk must be accessible to and usable by individuals with disabilities.

Refuge islands enhance pedestrian comfort by reducing effective walking distances and pedestrian exposure to traffic. Islands should be a minimum of 6 ft [1.8m] wide to afford refuge to people in wheelchairs. A minimum 5 ft [1.5m] wide by 6 ft [1.8m] long curb ramp should be cut through the island for pedestrian passage. Install curb ramps with a minimum 5 ft x 5 ft [1525 mm x 1525 mm] landing in the island if room allows, see Figure 2-10. Curb ramps and crosswalks must be aligned behind the nose of the median island to provide adequate refuge. If the crossing is very wide and the flashing don't walk time is not adequate to cross the entire width of the road, then push buttons are required and must be incorporated in the design.
Figure 2-8. Curb Ramps and Landings
Figure 2-9. Sidewalks at Driveway Aprons.
Curb Ramps and Landings. Curb ramps must be provided in conjunction with each project where the following types of work will be performed:

- Reconstruction, rehabilitation and resurfacing projects, including overlays, where a barrier exists to a sidewalk or a prepared surface for pedestrian use,
- Construction of curbs, curb and gutter, and/or sidewalks,
- Installation of traffic signals which include pedestrian signals, and
- Installation of pavement markings for pedestrian crosswalk.

A sidewalk curb ramp and level landing will be provided wherever a public sidewalk crosses a curb or other change in level. The maximum grade for curb ramps is 8.3 percent. The maximum cross...
slope for curb ramps is 2 percent. Flatter grades and slopes should be used where possible and to allow for construction tolerances and to improve accessibility. The preferred width of curb ramps is 5 ft [1.5m] and the minimum width is 4 ft [1.2m], exclusive of flared sides. Where a side of a curb ramp is contiguous with a public sidewalk or walking surface, it will be flared with a slope of 10 percent maximum, measured parallel to the curb.

Where a perpendicular or directional curb ramp is provided, a landing must be provided at the top of the ramp run. The slope of the landing will not exceed 2 percent in any direction. The landing should have a minimum clear dimension of 5 ft x 5 ft [1.5m x 1.5m] square or accommodate a 5 ft [1.5m] diameter circle and will connect to the continuous passage in each direction of travel as shown in Figure 2-8. Landings may overlap with other landings.

Where a parallel curb ramp is provided (i.e., the sidewalk ramps down to a landing at street level) a minimum 5 ft x 5 ft [1.5m x 1.5m] landing should be provided at the entrance to the street.

The bottom of a curb ramp run shall be wholly contained within the markings of the crosswalk. There shall be a minimum 4 ft x 4 ft [1.2m x 1.2m] maneuvering space wholly contained within the crosswalk, whether marked or unmarked and outside the path of parallel vehicular traffic.

Curb inlets, manhole covers, grates, and obstructions should not be located within the curb ramp, maneuvering area, or landing.

The standard sheet PED may be referenced for additional information on the configuration of curb ramps.

**Cross Slope.** Sidewalk cross slope will not exceed 1:50 (2 percent). Due to construction tolerances, it is recommended that sidewalk cross slopes be shown in the plans at 1.5 percent to avoid exceeding the 2 percent limit when complete. Cross slope requirements also apply to the continuation of the pedestrian route through the cross walk. 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 as shown in Figure 2-9. Where the ramp sidewalk must be sloped to cross a driveway, the designer is encouraged to use a running slope of 5 percent or less on the sloping portions of the sidewalk to avoid the need for level landings and handrails.

**Street Furniture.** Special consideration should be given to the location of street furniture (items intended for use by the public such as benches, public telephones, bicycle racks, and parking meters). Generally, a clear space at least 2.5 ft x 4 ft [760 mm x 1.2m] with a maximum slope of 2 percent must be provided and positioned to allow for either forward or parallel approach to the element in compliance with the Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way (PROWAG) and Texas Accessibility Standards (TAS). The clear ground space must have an accessible connection to the sidewalk and must not encroach into the minimum sidewalk width. Pedestrian push buttons and other operable parts shall be placed within specified reach ranges of a clear space.
Curb and Curb with Gutters

Curb designs are classified as vertical or sloping. Vertical curbs are defined as those having a vertical or nearly vertical traffic face 6 inches [150 mm] or higher. Vertical curbs are intended to discourage motorists from deliberately leaving the roadway. Sloping curbs are defined as those having a sloping traffic face 6 inches [150 mm] or less in height. Sloping curbs can be readily traversed by a motorist when necessary. A preferable height for sloping curbs at some locations may be 4 inches [100 mm] 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 urban areas. They should not be used in connection with the through, high-speed 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 sloping type.

Roadside Design

Of particular concern to the design engineer is the number of single-vehicle, run-off-the-road accidents which occur even on the safest facilities. About one-third of all highway fatalities are associated with crashes of this nature. The configuration and condition of the roadside greatly affect the extent of damages and injuries for these crashes.

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

- A “forgiving” roadside should be provided, free of unyielding obstacles including landscaping, drainage facilities that create obstacles, steep slopes, utility poles, etc. For adequate safety, it is desirable to provide an unencumbered roadside recovery area that is as wide as practicable for the specific highway and traffic conditions.

- For existing highways, treatment of obstacles should be considered in the following order:
  - Remove the obstacle.
  - Redesign the obstacle so that it can be safely traversed.
  - Relocate the obstacle to a point where it is less likely to be struck.
  - Make the obstacle breakaway.
  - Apply a cost-effective device to provide for redirection (longitudinal barrier) or severity reduction (impact attenuators). Barrier should only be used if the barrier is less of an obstacle than the obstacle it would protect, or if the cost of otherwise safety treating the obstacle is prohibitive.
  - Delineate the obstacle.

- Use of higher than minimum design standards result in a driver environment which is fundamentally safer because it is more likely to compensate for driver errors. Frequently, a design,
including sight distances greater than minimum, flattened slopes, etc., costs little more over the life of a project and increases safety and usefulness substantially.

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

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

Slopes and Ditches

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

**Slope Rates.** The path that an out-of-control vehicle follows after it leaves the traveled portion of the roadway is related to a number of factors such as driver capabilities, slope rates, and vehicular speed. Crash data indicates that approximately 75 percent of reported encroachments do not exceed a lateral distance of 30 ft [9 m] from the travel lane edge where roadside slopes are 1V:6H or flatter - slope rates that afford drivers significant opportunity for recovery. Crash test data further indicates that steeper slopes (up to 1V:3H) are negotiable by drivers; however, recovery of vehicular control on these steeper slopes is less likely. Recommended clear zone width associated with these slopes are further discussed in Clear Zone.

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

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

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

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

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

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

Roadside drainage ditches should be of sufficient width and depth to handle the design run-off and should be at least 6 inches [150 mm] below the subgrade crown to insure stability of the base course. For additional information, see [Drainage Facility Placement](#).

**Lateral Offset to Obstructions**

It is generally desirable that there be uniform clearance between traffic and roadside features such as bridge railings, parapets, retaining walls, and roadside barriers. In an urban environment, right of way is often limited and is characterized by sidewalks, enclosed drainage, numerous fixed objects (e.g., signs, utility poles, luminaire supports, fire hydrants, sidewalk furniture, etc.), and traffic making frequent stops. Uniform alignment enhances highway safety by providing the driver with a certain level of expectation, thus reducing driver concern for and reaction to those objects. The distance from the edge of the traveled way, beyond which a roadside object will not be perceived as an obstacle and result in a motorist’s reducing speed or changing vehicle position on the roadway, is called the lateral offset. This lateral offset to obstructions helps to:

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

As a minimum, as long as the obstruction is located beyond the recommended paved shoulder of a roadway, it will have minimum impact on driver speed or lane position and meet the lateral offset
requirement. Where a curb is present, the lateral offset is measured from the face of curb and shall be a minimum of 1.5 ft [0.5 m]. A minimum of 1 ft [0.3 m] lateral offset should be provided from the toe of barrier to the edge of traveled way.

**Clear Zone**

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

### Table 2-12: Clear Zones

<table>
<thead>
<tr>
<th>Location</th>
<th>Functional Classification</th>
<th>Design Speed (mph)</th>
<th>Avg. Daily Traffic</th>
<th>Clear Zone Width (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural</td>
<td>Freeways</td>
<td>All</td>
<td>All</td>
<td>Minimum 30 (16 for ramps)</td>
</tr>
<tr>
<td>Rural</td>
<td>Arterial</td>
<td>All</td>
<td>0 - 750</td>
<td>16 30</td>
</tr>
<tr>
<td>Rural</td>
<td>Collector</td>
<td>≥ 50</td>
<td>All</td>
<td>10 30</td>
</tr>
<tr>
<td>Rural</td>
<td>Collector</td>
<td>≤ 45</td>
<td>All</td>
<td>10 30</td>
</tr>
<tr>
<td>Rural</td>
<td>Local</td>
<td>All</td>
<td>All</td>
<td>10 30</td>
</tr>
<tr>
<td>Suburban</td>
<td>All</td>
<td>All</td>
<td>&lt;8,000</td>
<td>10 10^6 20^6</td>
</tr>
<tr>
<td>Suburban</td>
<td>All</td>
<td>All</td>
<td>8,000 - 12,000</td>
<td>10^6 20^6</td>
</tr>
<tr>
<td>Suburban</td>
<td>All</td>
<td>All</td>
<td>12,000 - 16,000</td>
<td>10^6 25^6</td>
</tr>
<tr>
<td>Suburban</td>
<td>All</td>
<td>All</td>
<td>&gt;16,000</td>
<td>20^6 30^6</td>
</tr>
<tr>
<td>Urban</td>
<td>Freeways</td>
<td>All</td>
<td>All</td>
<td>30 (16 for ramps)</td>
</tr>
<tr>
<td>Urban</td>
<td>All (Curbed)</td>
<td>≥ 50</td>
<td>All</td>
<td>Use above suburban criteria insofar as available border width permits.</td>
</tr>
<tr>
<td>Urban</td>
<td>All (Curbed)</td>
<td>≤ 45</td>
<td>All</td>
<td>4 from curb face 6</td>
</tr>
<tr>
<td>Urban</td>
<td>All (Uncurbed)</td>
<td>≥ 50</td>
<td>All</td>
<td>Use above suburban criteria.</td>
</tr>
<tr>
<td>Urban</td>
<td>All (Uncurbed)</td>
<td>≤ 45</td>
<td>All</td>
<td>10 25</td>
</tr>
</tbody>
</table>
The clear zone values shown in Table 2-12 are measured from the edge of travel lane. These are appropriate design values for all cut sections (see Drainage Facility Placement), for cross sectional design of ditches within the clear zone area) and for all fill sections with side slopes 1V:4H or flatter. It should be noted that, while a 1V:4H slope is acceptable, that a 1V:6H or flatter slope is preferred for both errant vehicle performance and slope maintainability. It is therefore preferable to provide an obstacle-free area of 10 ft[3.0m] beyond the toe of steep side slopes even when this area is outside the clear zone.

<table>
<thead>
<tr>
<th>Table 2-12: Clear Zones</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Because of the need for specific placement to assist traffic operations, devices such as traffic signal supports, railroad signal/warning device supports, and controller cabinets are excluded from clear zone requirements. However, these devices should be located as far from the travel lanes as practical. Other non-breakaway devices should be located outside the prescribed clear zone or these devices should be protected with barrier.</td>
</tr>
<tr>
<td>2 Average ADT over project life, i.e., 0.5 (present ADT plus future ADT). Use total ADT on two-way roadways and directional ADT on one-way roadways.</td>
</tr>
<tr>
<td>3 Without barrier or other safety treatment of appurtenances.</td>
</tr>
<tr>
<td>4 Measured from edge of travel lane for all cut sections and for all fill sections where side slopes are 1V:4H or flatter. Where fill slopes are steeper than 1V:4H it is desirable to provide a 10 ft area free of obstacles beyond the toe of slope.</td>
</tr>
<tr>
<td>5 Desirable, rather than minimum, values should be used where feasible.</td>
</tr>
<tr>
<td>6 Purchase of 5 ft or less of additional right-of-way strictly for satisfying clear zone provisions is not required.</td>
</tr>
</tbody>
</table>
Overview

This section contains information on the following topics:

- Design Treatment of Cross Drainage Culvert Ends
- Parallel Drainage Culverts
- Side Ditches

Introduction

In designing drainage systems, the primary objective is to properly accommodate surface run-off along and across highway right-of-way through the application of sound hydraulic principles. Consideration must also be given to incorporating safety into the design of drainage appurtenances. The best design would efficiently accommodate drainage and be traversable by an out-of-control vehicle without rollover or abrupt change in speed.

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

- Design or treat drainage appurtenances so that they will be traversable by a vehicle without rollover or abrupt change in speed.
- Locate appurtenances a sufficient distance, consistent with traffic volume, from the travel lanes so as to reduce the likelihood of accidental collision.
- 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 [80 km/h] or more and with rural type (uncurbed) cross sections. Where reference is made to clear zone requirements in these guidelines, see Table 2-12: Clear Zones and the discussions regarding Slopes and Ditches, Roadside Design, and Clear Zone. Desirable values for clear zone width should generally be used and minimum clear zone widths applied where unusual conditions are encountered. Site visits may be appropriate to ascertain terrain conditions and debris potential before arriving at design decisions for.

Designers should address and resolve culvert end treatment issues with involved parties 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 project concept conference with the appropriate entities prior to in-depth development of Plans, Specifications, and Estimates (P.S.&E.).
Chapter 2 — Basic Design Criteria

Section 7 — Drainage Facility Placement

Design Treatment of Cross Drainage Culvert Ends

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

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

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

Small Pipe Culverts. A small pipe culvert is defined as a single round pipe with 36 inches [900 mm] or less diameter, or multiple round pipes each with 30 inches [750 mm] or less diameter, each oriented on normal skew. (Note: For arch pipes, use span dimension instead of diameter.)

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

<table>
<thead>
<tr>
<th>Skew (degree)</th>
<th>Single Pipe (in.)</th>
<th>Multiple Pipe (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>30</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>45</td>
<td>24</td>
<td>21</td>
</tr>
</tbody>
</table>

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 pipe ends should be sloped at a rate of 1V:3H or flatter and should match side slope rate thereby providing a flush, traversable safety treatment. Single box culverts on normal skew with spans of 36 inches [900 mm] or less may be effectively safety treated just as small pipes (open, match 1V:3H or flatter slope).

When vulnerable to run-off-the-road vehicles (i.e., unshielded by barrier), sloped ends should be provided on small pipe culverts regardless of culvert end location with respect to 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. The resultant culvert with sloped end is both safe and inexpensive.

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. Headwalls should not be used.
In summary, whether a small pipe culvert is new or existing, sloped open ends should normally be used. Terrain in the vicinity of the culvert ends should be smooth and free of fixed objects.

**Intermediate Size Single Box Culverts and (Single and Multiple) Pipe Culverts.** An intermediate size pipe culvert is defined as a single round pipe with more than 36 inches [900 mm] diameter or multiple round pipes each with more than 30 inches [750 mm] diameter but having maximum diameter of 60 inches [1,500 mm]. (Note: 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. [1,500 mm]. Cross sectional area of the single box or individual pipe normally should not exceed 25 ft² [2.3 m²].

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

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

Sloped end treatments with safety pipe runners are preferred from a safety standpoint and are generally cost effective for both new and existing intermediate size culverts, regardless of end location with respect to clear zone criteria. These end treatments should be sloped at a rate of 1V:3H or flatter and should match the side slope rate thereby providing a flush, traversable safety treatment. Length of new culverts should be governed by the locations of the side slope plane/culvert intercepts rather than by clear zone. Terrain in the vicinity of 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 warranted for certain culvert end offsets and traffic volumes as shown in Table 2-12: Clear Zones. Where an improved design is warranted using Table 2-13, the removal of headwalls and installation of sloped ends with safety pipe runners is the preferred safety treatment.

In certain situations (e.g., culvert skew exceeds 15 degrees, severe debris problems, etc.) treatment with safety pipe runners may be impractical. For these conditions, locating intermediate size culvert ends to meet desirable clear zone values (see Table 2-12: Clear Zones) 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 inches [750 mm] (maximum) center to center thereby can minimize debris problems.

**Multiple Box Culverts and Large Single Pipes or Boxes.** Multiple box culverts are defined as those with more than one barrel and a total opening (i.e., distance) of 20 ft [6.1 m] or less between extreme inside faces as measured along the highway centerline. Large single pipes or single boxes are defined as those with diameter or height exceeding 5 ft [1,500 mm] or cross sectional area exceeding 25 ft² [2.3 m²].
From a safety standpoint alone, treatment is in the following priority for both new and existing installations:

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. First, multiple box culverts accommodate significantly greater flow quantities than single box or pipe culverts and often a defined channel crosses the highway right-of-way. 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, but lower priority, culvert end treatment be selected.

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

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

**Bridge Class Drainage Culverts.** Bridge class culverts are defined as those having an opening (i.e., distance) of more than 20 ft [6.1 m] between the extreme inside faces as measured along the highway centerline. Bridge class culverts require protection whether they are inside or outside the clear zone, exceptions to this requirement can be obtained by approval of a Design Exception or Design Waiver Request by the Bridge Division (see the Bridge Railing Manual for specific exception criteria).

Recommended treatment options are in the following priority:

1. Safety treat culvert ends.
2. Shield with appropriate barrier or attenuator. Table 2-14 provides guidelines for installing guardrails and bridge rails.
See discussion under “Multiple Box Culverts” for considerations when deciding on whether to safety treat or shield the culvert ends.

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

<table>
<thead>
<tr>
<th>Depth of Cover</th>
<th>Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 9 in.</td>
<td>Bridge Railing</td>
</tr>
<tr>
<td>9 in. but less than 36 in.</td>
<td>Steel post welded to base plate and bolted to culvert ceiling (Low-fill culvert post option on Guardrail standard)</td>
</tr>
<tr>
<td>36 in. or more(^1)</td>
<td>Standard Guardrail</td>
</tr>
</tbody>
</table>

\(^1\) Refer to Bridge Division for further guidance.

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

Where guardrail is carried across a bridge class culvert, steep side slopes should be positioned to provide for lateral support of the guardrail, as shown in Figure 2-11.
Figure 2-11. Use of Guardrail at Culverts.
Parallel Drainage Culverts

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

- Within the clear zone, there should be no culvert headwalls or vertical ends. Outside the clear zone, single pipe ends preferably should be sloped although not required.

- Where used, sloped pipe ends should be at a rate of 1V:6H or flatter. The sloping end may be terminated and a vertical section introduced at the top and bottom of the partial pipe section as shown in Figure 2-12.

- 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. [750 mm] in diameter pipe ends are located within the clear zone, safety pipe runners should be provided with a maximum slope steepness of 1V:6H with 1V:8H preferred. Typical details for a driveway, side road, or median crossover grate are shown in Figure 2-13. Cross pipes are not required on single, small (30 in. [750 mm] 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.

- 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

Figure 2-12. Use of Sloping Pipe Ends without Cross Pipes.

USE OF SLOPING PIPE ENDS WITH CROSS PIPES

Figure 2-13. Use of Sloping Pipe Ends with Grates.
Side Ditches

For side ditches, attention to cross section design can reduce the likelihood of serious injuries during vehicular encroachments. Ditches with the cross sectional characteristics defined in Table 2-15 are preferred and should especially be sought when ditch location is within the clear zone requirements. Where conditions dictate, such as insufficient existing right-of-way to accommodate the preferred 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 preferred ditch cross sections are provided.

<table>
<thead>
<tr>
<th>Preferred Maximum Back Slope (Vertical:Horizontal)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Given Front Slope</strong></td>
</tr>
<tr>
<td>1V:8H</td>
</tr>
<tr>
<td>1V:6H</td>
</tr>
<tr>
<td>1V:4H</td>
</tr>
<tr>
<td>1V:3H</td>
</tr>
</tbody>
</table>

Ditches that include retards to control erosion should be avoided inside the clear zone requirements and should be located as far from the travel lanes as practical unless the retardant is a rock filter dam with side slopes of 1V:6H or flatter. Non-traversable catch or stilling basins should also be located outside the clear zone requirements.
Section 8 — Roadways Intersecting Department Projects

Roadways that intersect or tie into a facility which the department is constructing must improve or retain the existing geometry of the intersecting roadway, or meet the design criteria for the roadway classification of the intersecting road. When a non-state intersecting road needs to adapt its alignment to the state's road improvements, there is no need for a design exception or design waiver as long as the non-state road’s geometry retains or exceeds its existing geometry. Existing geometry will include all cross sectional elements. The definition of intersecting roadways excludes driveways.

When the intersecting road is modified and changed to tie into a TxDOT facility due to the request by the county or city then it must follow TxDOT design criteria for local streets. If the criteria is not met, a design exception is required.
Chapter 3 — New Location and Reconstruction (4R) Design Criteria

Contents:

Section 1 — Overview
Section 2 — Urban Streets
Section 3 — Suburban Roadways
Section 4 — Two-Lane Rural Highways
Section 5 — Multi-Lane Rural Highways
Section 6 — Freeways
Section 7 — Freeway Corridor Enhancements
Section 8 — Texas Highway Freight Network (THFN)
Section 1 — Overview

Introduction

This chapter presents guidelines that are applicable to all new location and reconstruction projects for several different functional classes of roadways, as well as Freeways and the Texas Highway Freight Network (THFN):

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

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

Departures from these guidelines are governed in Design Exceptions, Design Waivers, Design Variances, and Texas Highway Freight Network (THFN) Design Deviations, Chapter 1.
Section 2 — Urban Streets

Overview

The term “Urban Street” as used in this chapter refers to roadways in developed areas that provide access to abutting property as well as movement of vehicular traffic. Access to these facilities is controlled through driveway locations, medians, and intersection of other roadways.

Level of Service

Urban streets and their auxiliary facilities should be designed for level of service B as defined in the Highway Capacity Manual. Heavily developed urban areas may necessitate the use of level of service D. The class of urban facility should be carefully selected to provide the appropriate level of service. For more information regarding level of service as it relates to facility design, see Service Flow Rate under subhead Traffic Volume in Chapter 2.

Basic Design Features

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

- Table 3-1: Geometric Design Criteria for Urban Streets
- Medians
- Median Openings
- Borders
- Berms
- Grade Separations and Interchanges
- Right-of-Way Width
- Intersections
- Speed Change Lanes
- Horizontal Offsets
- Bus Facilities.

Table 3-1 shows tabulated basic geometric design criteria for urban arterial, collector, and local streets. The basic design criteria shown in this table reflects minimum and desirable values applicable to new location, reconstruction or major improvement projects (such as widening to provide additional lanes).
### Table 3-1: Geometric Design Criteria for Urban Streets

<table>
<thead>
<tr>
<th>Item</th>
<th>Functional Class</th>
<th>Desirable</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design Speed (mph)</strong></td>
<td>All</td>
<td>Up to 60</td>
<td>30</td>
</tr>
<tr>
<td><strong>Minimum Horiz. Radius</strong></td>
<td>All</td>
<td>See Tables 2-3 and 2-4, Figure 2-3</td>
<td></td>
</tr>
<tr>
<td><strong>Maximum Gradient (%)</strong></td>
<td>All</td>
<td>See Table 2-11</td>
<td></td>
</tr>
<tr>
<td><strong>Width of Travel Lanes (ft)</strong></td>
<td>Arterial</td>
<td>12</td>
<td>11(^1)</td>
</tr>
<tr>
<td><strong>Curb Parking Lane Width (ft)</strong></td>
<td>Collector</td>
<td>12</td>
<td>10(^2)</td>
</tr>
<tr>
<td><strong>Maximum Gradient (%)</strong></td>
<td>Arterial</td>
<td>10</td>
<td>7(^5)</td>
</tr>
<tr>
<td><strong>Shoulder Width (ft), Uncurbed Urban Streets</strong></td>
<td>Arterial</td>
<td>10</td>
<td>4(^6,11)</td>
</tr>
<tr>
<td><strong>Width of Speed Change Lanes (ft)</strong></td>
<td>Arterial</td>
<td>11-12</td>
<td>10(^7)</td>
</tr>
<tr>
<td><strong>Offset to Face of Curb (ft)</strong></td>
<td>Arterial</td>
<td>11-12</td>
<td>9</td>
</tr>
<tr>
<td><strong>Median Width</strong></td>
<td>Arterial</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td><strong>Border Width (ft)</strong></td>
<td>Arterial</td>
<td>20</td>
<td>15</td>
</tr>
<tr>
<td><strong>Right-of-Way Width</strong></td>
<td>Arterial</td>
<td>Variable (^1)</td>
<td></td>
</tr>
<tr>
<td><strong>Clear Sidewalk Width (ft)</strong></td>
<td>All</td>
<td>6-8(^8)</td>
<td>5</td>
</tr>
<tr>
<td><strong>On-Street Bicycle Lane Width</strong></td>
<td>All</td>
<td>See Chapter 6, Bicycle Facilities</td>
<td></td>
</tr>
<tr>
<td><strong>Superelevation</strong></td>
<td>All</td>
<td>See Chapter 2, Superelevation</td>
<td></td>
</tr>
<tr>
<td><strong>Clear Zone Width</strong></td>
<td>All</td>
<td>See Table 2-12</td>
<td></td>
</tr>
<tr>
<td><strong>Vertical Clearance for New Structures (ft)</strong></td>
<td>All</td>
<td>16.5(^{1/2})</td>
<td>16.5(^{2/3},11)</td>
</tr>
<tr>
<td><strong>Turning Radii</strong></td>
<td>-</td>
<td>See Chapter 7, Minimum Designs for Truck and Bus Turns</td>
<td></td>
</tr>
</tbody>
</table>
### Design Criteria

#### Section 2 — Urban Streets

1. In highly restricted locations or locations with few trucks and speeds less than or equal to 40 mph, 10 ft permissible.
2. In industrial areas 12 ft usual, and 11 ft minimum for restricted R.O.W. conditions. In non-industrial areas, 10 ft minimum.
3. In residential areas, 9 ft minimum.
4. Where there is no demand for use as a future through lane, 8 ft minimum.
5. In commercial and industrial areas, 8 ft minimum.
6. Where only minimum width is provided, it should be fully surfaced. Where desirable width is provided, partial (not less than minimum width) surfacing or full width surfacing may be provided at the option of the designer.
7. Right-of-way width is a function of roadway elements as well as local conditions.
8. Applicable for commercial areas, school routes, or other areas with concentrated pedestrian traffic.
9. Exceptional cases near as practical to 16.5 ft but never less than 14.5 ft. Existing structures that provide at least 14 ft may be retained.
10. Cross slopes, ramps, and sidewalks shall be in compliance with the Americans with Disabilities Act Accessibility Guidelines and the Texas Accessibility Standards. See Chapter 2, Curb and Curb and Gutters and Sidewalks and Pedestrian Elements.
11. A 5 ft minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated except on off-system facilities with less than 400 ADT. See Ch. 2, Section 6 for additional information.
12. Additional vertical clearance requirements will apply to roadways on the THFN for projects Let on September 1, 2020 or later. See Ch. 3, Section 8 for specific requirements.

#### Table 3-1: Geometric Design Criteria for Urban Streets

<table>
<thead>
<tr>
<th>Item</th>
<th>Functional Class</th>
<th>US Customary</th>
<th>Metric</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Item</strong></td>
<td><strong>Desirable</strong></td>
<td><strong>Minimum</strong></td>
<td></td>
</tr>
<tr>
<td>Design Speed (km/h)</td>
<td>All</td>
<td>Up to 100</td>
<td>50</td>
</tr>
<tr>
<td>Minimum Horiz. Radius</td>
<td>All</td>
<td>See Tables 2-3 and 2-4, Figure 2-3</td>
<td></td>
</tr>
<tr>
<td>Maximum Gradient (%)</td>
<td>All</td>
<td>See Table 2-11</td>
<td></td>
</tr>
<tr>
<td>Stopping Sight Distance</td>
<td>Arterial</td>
<td>3.6</td>
<td>3.31</td>
</tr>
<tr>
<td></td>
<td>Collector</td>
<td>3.6</td>
<td>3.02</td>
</tr>
<tr>
<td></td>
<td>Local</td>
<td>3.3-3.6</td>
<td>3.023</td>
</tr>
<tr>
<td>Curb Parking Lane Width (m)</td>
<td>Arterial</td>
<td>3.6</td>
<td>3.04</td>
</tr>
<tr>
<td></td>
<td>Collector</td>
<td>3.0</td>
<td>2.15</td>
</tr>
<tr>
<td></td>
<td>Local</td>
<td>2.7</td>
<td>2.15</td>
</tr>
<tr>
<td>Shoulder Width (m), Uncurbed Urban Streets</td>
<td>Arterial</td>
<td>3.0</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Collector</td>
<td>2.4</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>Local</td>
<td>--</td>
<td>0.6</td>
</tr>
<tr>
<td>Width of Speed Change Lanes (m)</td>
<td>Arterial and Collector</td>
<td>3.3-3.6</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>Local</td>
<td>3.0-3.6</td>
<td>2.7</td>
</tr>
<tr>
<td>Offset to Face of Curb (m)</td>
<td>All</td>
<td>0.6</td>
<td>0.3</td>
</tr>
</tbody>
</table>
For minor rehabilitation projects where no additional lanes are proposed, existing curbed cross sections should be compared with the design criteria in Table 3-1 to determine the practicality and economic feasibility of minor widening to meet the prescribed standards. Where only minimal widening is required to conform with a standard design, it is often cost effective to retain the existing street section, thereby sparing the cost of removing and replacing concrete curb and gutter and curb inlets. For these type projects, Resurfacing, Restoration, and Rehabilitation (3R) guidelines are usually applicable, see Chapter 4.
Medians

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

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

Medians used on urban streets include the following types:

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

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

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

Where ADT exceeds 20,000 vehicles per day or where development is occurring, and volumes are increasing and are anticipated to reach this level, and the demand for mid-block turns is high, a raised median design should be considered. For these conditions, a raised median may improve safety by separating traffic flows and controlling left-turn and crossing maneuvers. The use of raised medians should be discouraged where the roadway cross-section is too narrow for U-turns.

For median left turn lanes at intersections, a median width of 16 ft [4.8 m] (12 ft [3.6 m] lane plus a 4 ft [1.2 m] divider) is recommended to accommodate a single left turn lane. A median width of 18 ft (5.5 m) is required if pedestrian refuge area is needed, see Figure 2-10. For maintenance considerations in preventing recurring damage to the divider, the divider should be at least 2 ft [0.6 m] wide. If pedestrians are expected to cross the divider, then the divider should be a minimum of 6 ft [1.8 m] wide in order to accommodate a cut-through landing or refuge area that is at least 5 ft x 6 ft [1.5 m x 1.8 m]. See Dual Left-Turn Lanes for additional median width discussion.

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

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

**Two-Way Left-Turn Lanes.** Two-way left-turn lanes (TWLTL) are flush medians that may be used for left turns by traffic from either direction on the street. The TWLTL is appropriate where there is a high demand for mid-block left turns, such as areas with (or expected to experience) moderate or intense strip development. Used appropriately, the TWLTL design has improved the safety and operational characteristics of streets as demonstrated through reduced travel times and crash rates. The TWLTL design also offers added flexibility since, during spot maintenance activities, a travel lane may be barricaded with through traffic temporarily using the median lane.

Recommended median lane widths for the TWLTL design are as shown in Table 3-2. In applying these criteria on new location projects or on reconstruction projects where widening necessitates the removal of exterior curbs, the median lane width should not be less than 12 ft [3.6 m], and preferably the desirable value shown in Table 3-2. Minimum values shown in Table 3-2 are appropriate for restrictive right-of-way projects and improvement projects where attaining the desirable width would necessitate removing and replacing exterior curbing to gain only a small amount of roadway width.

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

<table>
<thead>
<tr>
<th>Design Speed Mph</th>
<th>Width of TWLTL - ft</th>
<th>Design Speed Km/h</th>
<th>Width of TWLTL – m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Desirable</td>
<td>Minimum</td>
<td>Desirable</td>
</tr>
<tr>
<td>Less than or equal to 40</td>
<td>12 - 14</td>
<td>11</td>
<td>Less than or equal to 60</td>
</tr>
<tr>
<td>45 - 50</td>
<td>14</td>
<td>12</td>
<td>70 - 80</td>
</tr>
<tr>
<td>Greater than 50</td>
<td>16</td>
<td>14</td>
<td>Greater than 80</td>
</tr>
</tbody>
</table>

Criteria for the potential use of a TWLTL for urban streets are as follows:

- Future ADT volume of 3,000 vehicles per day for an existing two-lane urban street, 6,000 vehicles per day for an existing four-lane urban street, or 10,000 vehicles per day for an existing six-lane urban street, and
- Side road plus driveway density of 20 or more entrances per mile [12 or more entrances per kilometer].

When the above two conditions are met, the site should be considered suitable for the use of a TWLTL. For ADT volumes greater than 20,000 vehicle per day or where development is occurring,
and volumes are increasing and are anticipated to reach this level, a raised median design should be considered. All cross sections should be evaluated for pedestrian crossing capabilities.

**Median Openings**

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 signal detection loops to operate without false calls. A directional opening can be used to limit the number and type of conflict. Figures 3-1 illustrates the different options for the design of a directional median opening.

*Figure 3-1. Types of Directional Openings.*
Borders

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

Berms

There are two different types of berms typically used on urban streets. One type of berm is constructed as a narrow shelf or path. This type is typically used to provide a flush grade behind a curb to accommodate the possible future installation of sidewalks.

Another type of berm is constructed as a raised mound to facilitate drainage or for landscaping purposes. When this type of berm is constructed, it is desirable that the berm be placed outside of the clear zone. If this is not practical, care should be taken to ensure that the slopes and configurations used meet the clear zone requirements as discussed in Slopes and Ditches in Chapter 2.

Grade Separations and Interchanges

Although grade separations and interchanges are not often provided on urban streets, they may be the only means available for providing sufficient capacity at critical intersections. Normally, a grade separation is part of an interchange (except grade separations with railroads); it is usually the diamond type where there are four legs. Locations considered include high volume intersections and where terrain conditions favor separation of grades.

The entire roadway width of the approach, including parking lanes or shoulders if applicable, 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, diamond ramps may have lengths 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 lengthy section of arterial street. In these cases, the street assumes the operating characteristics and appearance of a freeway. In this regard, where right-of-way availability permits, it may be appropriate to eliminate the relatively few crossings at-grade and control access by design (i.e., provide continuous frontage roads) in the interest of safety. It is not desirable, however, to intermix facility types by providing intermittent sections of fully controlled and non-controlled access facilities.
Right-of-Way Width

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

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

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

Intersections

The number, design, and spacing of intersections influence the capacity, speed, and safety on urban streets. Capacity analysis of signalized intersections is one of the most important considerations in intersection design. Dimensional layout or geometric design considerations are closely influenced by traffic volumes and operational characteristics and the type of traffic control measures used.

Because of the space limitations and lower operating speeds on urban streets, curve radii for turning movements are less than for rural highway intersections. Curb radii of 15 ft [4.5 m] to 25 ft [7.5 m] permit passenger cars to negotiate right turns with little or no encroachment on other lanes. Where heavy volumes of trucks or buses are present, increased curb radii of 30 ft [9 m] to 50 ft [15 m] expedite turns to and from through lanes. Where combination tractor-trailer units are anticipated in significant volume, reference should be made to the material in Minimum Designs for Truck and Bus Turns, Chapter 7.

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

Figure 3-2 illustrates lines of sight for a vehicle entering an intersection.
Speed Change Lanes

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

- Deceleration length
- Storage length

**Left-Turn Deceleration Lanes.** Figure 3-3 illustrates the use of left-turn lanes on urban streets. A short symmetrical reverse curve taper or straight taper may be used. For median left-turn lanes, a minimum median width of 16 ft [4.8 m] (12 ft [3.6 m] lane width plus a 4 ft [1.2 m] divider) is recommended to accommodate a single left-turn lane. The absolute minimum median width is 14 ft [4.2 m]. Where dual left-turns are provided, a minimum median width of 28 ft. [8.5 m] is recommended (two 12 ft. [3.6 m] lanes plus a 4 ft. [1.2 m] divider). Where pedestrians may be present, the divider must be a minimum 6 ft. [1.8 m], see Figure 2-10. Where a raised divider extends into the pedestrian cross-walk, a cut-through that is a minimum of 5 ft. x 5 ft. [1.5 m x 1.5 m] must be provided, see Figure 2-10.
Table 3-3 provides recommended taper lengths, deceleration lengths, and storage lengths for left-turn lanes. These guidelines may also be applied to the design of right-turn lanes.

**Table 3-3: Lengths of Single Left-Turn Lanes on Urban Streets**

<table>
<thead>
<tr>
<th>Speed (mph)</th>
<th>Deceleration Length(^2) (ft.)</th>
<th>Taper Length (ft.)</th>
<th>Storage Length (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Signalized</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Calculated</td>
</tr>
<tr>
<td>30</td>
<td>160</td>
<td>50</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>35</td>
<td>215</td>
<td>50</td>
<td>See footnote 3</td>
</tr>
</tbody>
</table>
Table 3-3: Lengths of Single Left-Turn Lanes on Urban Streets

<table>
<thead>
<tr>
<th>Speed (mph)</th>
<th>Deceleration Length² (ft.)</th>
<th>Taper Length (ft.)</th>
<th>Storage Length (ft.)</th>
<th>Signalized</th>
<th>Non-Signalized</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Calculated</td>
<td>Minimum⁴</td>
</tr>
<tr>
<td>40</td>
<td>275</td>
<td>50</td>
<td>See footnote 3</td>
<td>100</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>45</td>
<td>345</td>
<td>100</td>
<td>See footnote 3</td>
<td>100</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>50</td>
<td>425</td>
<td>100</td>
<td>See footnote 3</td>
<td>100</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>55</td>
<td>510</td>
<td>100</td>
<td>See footnote 3</td>
<td>100</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>60</td>
<td>605</td>
<td>100</td>
<td>See footnote 3</td>
<td>100</td>
<td>See footnote 5</td>
</tr>
</tbody>
</table>

(Metric)

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Deceleration Length² (m)</th>
<th>Taper Length (m)</th>
<th>Storage Length (m)</th>
<th>Signalized</th>
<th>Non-Signalized</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Calculated</td>
<td>Minimum⁴</td>
</tr>
<tr>
<td>50</td>
<td>50</td>
<td>15</td>
<td>See footnote 3</td>
<td>30</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>65</td>
<td>85</td>
<td>15</td>
<td>See footnote 3</td>
<td>30</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>80</td>
<td>130</td>
<td>30</td>
<td>See footnote 3</td>
<td>30</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>95</td>
<td>185</td>
<td>30</td>
<td>See footnote 3</td>
<td>30</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>100</td>
<td>205</td>
<td>30</td>
<td>See footnote 3</td>
<td>30</td>
<td>See footnote 5</td>
</tr>
<tr>
<td>110</td>
<td>245</td>
<td>30</td>
<td>See footnote 3</td>
<td>30</td>
<td>See footnote 5</td>
</tr>
</tbody>
</table>

¹ The minimum length of a left-turn lane is the sum of the deceleration length plus queue storage. In order to determine the design length, the deceleration plus storage length must be calculated for peak and off-peak periods, the longest total length will be the minimum design length.

² See Deceleration Length discussion immediately following Table 3-3.

³ See Storage Length Calculations discussion immediately following Table 3-3A.

⁴ The minimum storage length shall apply when: 1) the required queue storage length calculated is less than the minimum length, or 2) there is no rational method for estimating the left-turn volume.

⁵ The calculated queue storage at unsignalized location using a traffic model or simulation model or by the following:

\[ L = \frac{V}{30}(2)(S) \]

Where: \( V/30 \) is the left-turn volume in a two-minute interval and other terms are as defined in the Storage Length Calculations discussion immediately following Table 3-3A.
Deceleration Length. Deceleration length assumes that moderate deceleration will occur in the through traffic lane and the vehicle entering the left-turn lane will clear the through traffic lane at a speed of 10 mph (15 km/h) slower than through traffic. Where providing this deceleration length is impractical, it may be acceptable to allow turning vehicles to decelerate more than 10 mph (15 km/h) before clearing the through traffic lane. See Table 3-3A.

Table 3-3A Deceleration Lengths for Speed Differentials Greater than 10 mph (15 km/h)

<table>
<thead>
<tr>
<th>US Customary (ft)</th>
<th>Metric (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed</td>
<td>Speed Differential*</td>
</tr>
<tr>
<td>(mph)</td>
<td>15 mph</td>
</tr>
<tr>
<td>30</td>
<td>110</td>
</tr>
<tr>
<td>35</td>
<td>160</td>
</tr>
<tr>
<td>40</td>
<td>215</td>
</tr>
<tr>
<td>45</td>
<td>275</td>
</tr>
<tr>
<td>50</td>
<td>345</td>
</tr>
<tr>
<td>55</td>
<td>425</td>
</tr>
</tbody>
</table>

* Speed differential = the difference between a turning vehicle when it clears the through traffic lane and speed of following through traffic. Clearance is considered to have occurred when the turning vehicle has moved laterally a sufficient distance (10 ft. [3 m]) so that a following through vehicle can pass without encroaching upon the adjacent through lane.

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 may be used:

\[ L = (V/N)(2)(S) \]

Where:
- \( L \) = storage length in feet (or meters)
- \( V \) = left-turn volume per hour, vph
- \( N \) = number of cycles
- \( 2 \) = 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 (or meters), per vehicle

| % of trucks | S (ft) | S (m) |
Dual Left-Turn Deceleration Lanes. For major signalized intersections where high peak hour left-turn volumes are expected, dual left-turn lanes should be considered. As with single left-turn lanes, dual left-turn lanes should desirably include length for deceleration, storage, and taper. Table 3-4 provides recommended lengths for dual left-turn lanes.

Table 3-4: Lengths of Dual Left-Turn Lanes on Urban Streets

<table>
<thead>
<tr>
<th>Speed (mph)</th>
<th>Deceleration Length (ft)</th>
<th>Taper Length (ft)</th>
<th>Storage Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Calculated<strong>3</strong></td>
</tr>
<tr>
<td>30</td>
<td>160</td>
<td>100</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>35</td>
<td>215</td>
<td>100</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>40</td>
<td>275</td>
<td>100</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>45</td>
<td>345</td>
<td>150</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>50</td>
<td>425</td>
<td>150</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>55</td>
<td>510</td>
<td>150</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>60</td>
<td>605</td>
<td>150</td>
<td>See footnote 3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Deceleration Length (m)</th>
<th>Taper Length (m)</th>
<th>Storage Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Calculated<strong>3</strong></td>
</tr>
<tr>
<td>50</td>
<td>50</td>
<td>30</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>65</td>
<td>85</td>
<td>30</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>80</td>
<td>130</td>
<td>45</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>95</td>
<td>185</td>
<td>45</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>100</td>
<td>205</td>
<td>45</td>
<td>See footnote 3</td>
</tr>
<tr>
<td>110</td>
<td>245</td>
<td>45</td>
<td>See footnote 3</td>
</tr>
</tbody>
</table>

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

![Diagram of Right-Turn Deceleration Lane](image)

*(See Table 3-3 for taper, deceleration and storage lengths)*

*Figure 3-4. Lengths of Right-Turn Deceleration Lanes.*

**Right-Turn Acceleration Lanes.** Acceleration lanes typically are not used on urban streets. See Section 5, [Figure 3-10](#) for acceleration distances and taper lengths if an acceleration lane may be necessary.

**Auxiliary Lanes on Crest Vertical Curves**

When an intersection or driveway is located beyond the crest of a vertical curve, the designer should check the driver’s view of the left-turn or right-turn lane as they approach the beginning of the taper. It is suggested that this preview time be at least two seconds. An auxiliary lane that is longer than the deceleration distance plus queue storage length may be a consideration, if practical, in these situations.
Horizontal Offsets

For low-speed streets, cross drainage culvert ends should be offset minimally 4 ft [1.2 m] from the back of curb or 4 ft [1.2 m] from outside edge of shoulder. The designer, however, should make the best use of available border width to obtain wide clearances. Sloped open ends may be used to effectively safety treat small culverts. Consideration should be given to future sidewalk needs.

Bus Facilities

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

This subsection includes the following topics:

- Bus lanes,
- Bus streets.

Bus Lanes

Bus lanes are usually used exclusively by buses; however, in some instances carpools, taxis, or turning vehicles may share the lane. They may be located along curbs or in street medians and may operate with, or counter to, automobile flow. For more information on bus lanes, see St. Jacques, Kevin and Herbert S. Levinson. *Operational Analysis of Bus Lanes on Arterials*, TCRP Report 26, TRB, National Research Council, Washington, DC (1997).

**Curb Bus Lanes (Normal Flow).** Curb bus lanes in the normal direction flow are usually in effect only during the peak periods. They are usually implemented in conjunction with removal of curb parking so that there is little adverse effect on existing street capacity. This type of operation may be difficult to enforce and may produce only marginal benefits to bus flow. In operation, right-turning vehicles conflict with buses.

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

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

The term “suburban roadway” refers to high-speed roadways that serve as transitions between low-speed urban streets and high-speed rural highways. Suburban roadways are typically 1 to 3 miles [1.6 to 4.8 kilometers] in length and have light to moderate driveway densities (approximately 10 to 30 driveways per mile [5 to 20 driveways per kilometer]). Because of their location, suburban roadways have both rural and urban characteristics. For example, these sections typically maintain high speeds (a rural characteristic) while utilizing curb and gutter to facilitate drainage (an urban characteristic). Consequently, guidelines for suburban roadways typically fall between those for rural highways and urban streets.

Basic Design Features

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

- **Access Control**,
- **Medians**,
- **Median Openings**,
- **Speed Change Lanes**,
- **Right of Way Width**,
- **Clear Zone**,
- **Borders**,
- **Grade Separations and Interchanges**,
- **Intersections**, and
- **Parking**.

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

<table>
<thead>
<tr>
<th>Item</th>
<th>Functional Class</th>
<th>Desirable</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed (mph)</td>
<td>All</td>
<td>60</td>
<td>50</td>
</tr>
<tr>
<td>Minimum Horizontal Radius</td>
<td>All</td>
<td>See Tables 2-3 and 2-4</td>
<td></td>
</tr>
<tr>
<td>Maximum Gradient (%)</td>
<td>All</td>
<td>See Table 2-11</td>
<td></td>
</tr>
<tr>
<td>Stopping Sight Distance</td>
<td>All</td>
<td>See Table 2-1</td>
<td></td>
</tr>
<tr>
<td>Width of Travel Lanes (ft.)</td>
<td>Arterial Collector</td>
<td>12</td>
<td>11(^1)</td>
</tr>
<tr>
<td></td>
<td>Collector</td>
<td>12</td>
<td>10(^2)</td>
</tr>
<tr>
<td>Curb Parking Lane Width (ft.)</td>
<td>All</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>Shoulder Width (ft.)</td>
<td>All</td>
<td>10</td>
<td>4(^7)</td>
</tr>
<tr>
<td>Width of Speed Change Lanes(^3) (ft.)</td>
<td>All</td>
<td>11-12</td>
<td>10</td>
</tr>
<tr>
<td>Offset to Face of Curb (ft.)</td>
<td>All</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Median Width</td>
<td>All</td>
<td>See Medians, Urban Streets</td>
<td></td>
</tr>
<tr>
<td>Border Width (ft.)</td>
<td>Arterial Collector</td>
<td>20</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Collector</td>
<td>20</td>
<td>15</td>
</tr>
<tr>
<td>Right-of-Way Width (ft.)</td>
<td>All</td>
<td>Variable(^4)</td>
<td></td>
</tr>
<tr>
<td>Sidewalk Width (ft.)</td>
<td>All</td>
<td>6-8(^5)</td>
<td>5</td>
</tr>
<tr>
<td>Superelevation</td>
<td>All</td>
<td>See Chapter 2, Superelevation Rate</td>
<td></td>
</tr>
<tr>
<td>Clear Zone</td>
<td>All</td>
<td>See Table 2-12</td>
<td></td>
</tr>
<tr>
<td>Vertical Clearance for New Structures (ft.)</td>
<td>All</td>
<td>16.5(^6), 16.5(^6), (^8)</td>
<td></td>
</tr>
<tr>
<td>Turning Radii</td>
<td>All</td>
<td>See Chapter 7, Minimum Designs for Truck and Bus Turns</td>
<td></td>
</tr>
</tbody>
</table>

\(^1\) In highly restricted locations, 10 ft. permissible.
\(^2\) In industrial areas 12 ft. usual, and 11 ft. minimum for restricted R.O.W. conditions. In non-industrial areas, 10 ft. minimum.
\(^3\) Applicable when right or left-turn lanes are provided.
\(^4\) Right-of-way width is a function of roadway elements as well as local conditions.
\(^5\) Applicable for commercial areas, school routes, or other areas with concentrated pedestrian traffic.
\(^6\) Exceptional cases near as practical to 16.5 ft. but never less than 14.5 ft. Existing structures that provide at least 14 ft. may be retained.
\(^7\) A 5 ft. minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated except on off-system facilities with less than 400 ADT. See Ch. 2, Section 6 for additional information.
\(^8\) Additional vertical clearance requirements will apply to roadways on the Texas Highway Freight Network (THFN) for projects Let on September 1, 2020 or later. See Ch. 3, Section 8 for specific requirements.
### Table 3-5: Geometric Design Criteria for Suburban Roadways

<table>
<thead>
<tr>
<th>Item</th>
<th>Functional Class</th>
<th>Desirable</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed (km/h)</td>
<td>All</td>
<td>100</td>
<td>80</td>
</tr>
<tr>
<td>Minimum Horizontal Radius</td>
<td>All</td>
<td>See Tables 2-3 and 2-4</td>
<td></td>
</tr>
<tr>
<td>Maximum Gradient (%)</td>
<td>All</td>
<td>See Table 2-11</td>
<td></td>
</tr>
<tr>
<td>Stopping Sight Distance</td>
<td>All</td>
<td>See Table 2-1</td>
<td></td>
</tr>
<tr>
<td>Width of Travel Lanes (m)</td>
<td>Arterial</td>
<td>3.6</td>
<td>3.3(^1)</td>
</tr>
<tr>
<td></td>
<td>Collector</td>
<td>3.6</td>
<td>3.0(^2)</td>
</tr>
<tr>
<td>Curb Parking Lane Width (m)</td>
<td>All</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>Shoulder Width (m)</td>
<td>All</td>
<td>3.0</td>
<td>1.2(^7)</td>
</tr>
<tr>
<td>Width of Speed Change Lanes(^3) (m)</td>
<td>All</td>
<td>3.3-3.6</td>
<td>3.0</td>
</tr>
<tr>
<td>Offset to Face of Curb (m)</td>
<td>All</td>
<td>0.6</td>
<td>0.3</td>
</tr>
<tr>
<td>Median Width</td>
<td>All</td>
<td>See Medians, Urban Streets</td>
<td></td>
</tr>
<tr>
<td>Border Width (m)</td>
<td>Arterial</td>
<td>6.0</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>Collector</td>
<td>6.0</td>
<td>4.5</td>
</tr>
<tr>
<td>Right-of-Way Width (m)</td>
<td>All</td>
<td>Variable(^4)</td>
<td></td>
</tr>
<tr>
<td>Sidewalk Width (m)</td>
<td>All</td>
<td>1.8-2.4(^5)</td>
<td>1.5</td>
</tr>
<tr>
<td>Superelevation</td>
<td>All</td>
<td>See Chapter 2, Superelevation Rate</td>
<td></td>
</tr>
<tr>
<td>Clear Zone</td>
<td>All</td>
<td>See Table 2-12</td>
<td></td>
</tr>
<tr>
<td>Vertical Clearance for New Structures (m)</td>
<td>All</td>
<td>5.0(^6)</td>
<td>5.0(^6,8)</td>
</tr>
<tr>
<td>Turning Radii</td>
<td>All</td>
<td>See Chapter 7, Minimum Designs for Truck and Bus Turns</td>
<td></td>
</tr>
</tbody>
</table>

---

1. In highly restricted locations, 3.0 m permissible.
2. In industrial areas 3.6 m usual, and 3.3 m minimum for restricted R.O.W. conditions. In non-industrial areas, 3.0 m minimum.
3. Applicable when right or left-turn lanes are provided.
4. Right-of-way width is a function of roadway elements as well as local conditions.
5. Applicable for commercial areas, school routes, or other areas with concentrated pedestrian traffic.
6. Exceptional cases near as practical to 5.0 m but never less than 4.4 m. Existing structures that provide at least 4.3 m may be retained.
7. A 1.5 m. minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated except on off-system facilities with less than 400 ADT. See Ch. 2, Section 6 for additional information.
8. Additional vertical clearance requirements will apply to roadways on the Texas Highway Freight Network (THFN) for projects Let on September 1, 2020 or later. See Ch. 3, Section 8 for specific requirements.
Access Control

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

Medians

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

**Raised Medians.** Raised medians with curbing are used on suburban arterials where it is desirable to control left-turn movements. These medians should be delineated with curbs of the mountable type. Raised medians are applicable on high-volume roadways with high demand for left turns. For additional guidelines regarding the installation of raised medians, see Raised Medians, *Urban Streets*.

**Two-Way Left-Turn Lanes.** The two-way left-turn lanes (TWLTL) is applicable on suburban roadways with moderate traffic volumes and low to moderate demands for left turns. For suburban roadways, TWLTL facilities should minimally be 14 ft [4.2 m] and desirably 16 ft [4.8 m] in width.

The desirable value of 16 ft [4.8 m] width should be used on new location projects or on reconstruction projects where widening necessitates the removal of exterior curbs. The “minimum” value of 14 ft [4.2 m] width is appropriate for restrictive right-of-way projects and improvement projects where attaining “desirable” median lane width would necessitate removing and replacing exterior curbing to gain only a small amount of roadway width.

Criteria for the potential use of a continuous TWLTL on a suburban roadway are as follows:

- Future ADT volume of 3,000 vehicles per day for an existing two-lane suburban roadway, 6,000 vehicles per day for an existing four-lane suburban roadway, or 10,000 vehicles per day for an existing six-lane suburban roadway, and

- Side road plus driveway density of 10 or more entrances per mile [6 or more entrances per kilometer].
When both conditions are met, the use of a TWLTL should be considered. For ADT volumes greater than 20,000 vehicle per day, or where development is occurring and volumes are increasing and are anticipated to reach this level, a raised median design should be considered.

Seven-lane cross sections should be evaluated for pedestrian crossing capabilities.

Median Openings

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

Speed Change Lanes

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

Right-of-Way Width

Similar to urban streets, the width of right-of-way for suburban roadways is influenced by traffic volume requirements, lane use, cost, extent of ultimate expansion, and land availability. Width is the summation of the various cross-sectional elements, including widths of travel and turning lanes, bicycle lanes, shoulders, median, sidewalks, sidewalk offsets, and borders.

Clear Zones

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

Borders

See Borders Urban Streets.

Grade Separations and Interchanges

See Grade Separations and Interchanges, Urban Streets.
Intersections

Due to high operating speeds (50 mph [80 km/h] or greater) on suburban roadways, curve radii for turning movements should equal that of rural highway intersections. Space restrictions due to right-of-way limitations in suburban areas, however, may necessitate reduction in the values given for rural highways. For additional information regarding intersection design, see Intersections Urban Streets.

Parking

Desirably, parking adjacent to the curb on suburban roadways should not be allowed.
Section 4 — Two-Lane Rural Highways

Overview

The general geometric features for two-lane rural highways are provided in this section and are summarized in the following tables and figures:

- **Figure 3-5**: Typical cross section
- **Table 3-6**: Minimum Design Speed for Rural Two-lane Highways: Minimum design speed
- **Table 3-7, Geometric Design Criteria for Rural Two-Lane Highways**: Basic design criteria and cross sectional elements
- **Table 3-8**: Width of Travel Lanes and Shoulders on Rural Two-lane Highways: Lane and shoulder widths
- **Table 3-9**: Minimum Structure Widths For Bridges to Remain in Place on Rural Two-lane Highways: Minimum structure widths that may remain in place.

Additional information on structure widths may be obtained in the [Bridge Design - LRFD](#) and the [Bridge Project Development Manual](#).

| Table 3-6: Minimum Design Speed for Rural Two-lane Highways |

<table>
<thead>
<tr>
<th>(US Customary)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Functional Class</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Arterial</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Collector</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Local&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
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</tbody>
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<table>
<thead>
<tr>
<th>(Metric)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Functional Class</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Arterial</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>
Table 3-6: Minimum Design Speed for Rural Two-lane Highways

(US Customary)

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>Terrain</th>
<th>Minimum Design Speed (mph) for future ADT of:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>&lt; 400</td>
</tr>
<tr>
<td>Collector</td>
<td>Level</td>
<td>80(^1)</td>
</tr>
<tr>
<td></td>
<td>Rolling</td>
<td>60(^{2/4})</td>
</tr>
<tr>
<td>Local(^3)</td>
<td>Level</td>
<td>60(^{2/4})</td>
</tr>
<tr>
<td></td>
<td>Rolling</td>
<td>50(^4)</td>
</tr>
</tbody>
</table>

\(^1\) A 40 mph [60 km/h] 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 [50 km/h] minimum design speed may be used where roadside environment or unusual design considerations dictate (e.g., significant horizontal curvature due to mountainous or hilly terrain).

\(^3\) Applicable only to off-system routes that are not functionally classified at a higher classification.

\(^4\) When determining applicable radii and superelevations, Tables 2-3, 2-4, 2-6, and 2-7 (for high speed and non-urban conditions) should be used even though these speeds are considered low-speed.

Table 3-7. Geometric Design Criteria for Rural Two-Lane Highways

(US Customary)

<table>
<thead>
<tr>
<th>Geometric Design Element</th>
<th>Functional Class</th>
<th>Reference or Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed</td>
<td>All</td>
<td>Table 3-6</td>
</tr>
<tr>
<td>Minimum Horizontal Radius</td>
<td>All</td>
<td>Table 2-3 and Table 2-4</td>
</tr>
<tr>
<td>Max. Gradient</td>
<td>All</td>
<td>Table 2-11</td>
</tr>
<tr>
<td>Stopping Sight Distance</td>
<td>All</td>
<td>Table 2-1</td>
</tr>
<tr>
<td>Width of Travel Lanes</td>
<td>All</td>
<td>Table 3-8</td>
</tr>
<tr>
<td>Width of Shoulders</td>
<td>All</td>
<td>Table 3-8</td>
</tr>
<tr>
<td>Vertical Clearance, New Structures</td>
<td>All</td>
<td>16.5 ft(^1), 2</td>
</tr>
<tr>
<td>Clear Zone</td>
<td>All</td>
<td>Table 2-12</td>
</tr>
<tr>
<td>Pavement Cross Slope</td>
<td>All</td>
<td>Chapter 2, Pavement Cross Slope</td>
</tr>
</tbody>
</table>

(Metric)

<table>
<thead>
<tr>
<th>Geometric Design Element</th>
<th>Functional Class</th>
<th>Reference or Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed</td>
<td>All</td>
<td>Table 3-6</td>
</tr>
</tbody>
</table>
Table 3-7. Geometric Design Criteria for Rural Two-Lane Highways

<table>
<thead>
<tr>
<th>Geometric Design Element</th>
<th>Functional Class</th>
<th>Reference or Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Horizontal Radius</td>
<td>All</td>
<td>Table 2-3 and Table 2-4</td>
</tr>
<tr>
<td>Max. Gradient</td>
<td>All</td>
<td>Table 2-11</td>
</tr>
<tr>
<td>Stopping Sight Distance</td>
<td>All</td>
<td>Table 2-1</td>
</tr>
<tr>
<td>Width of Travel Lanes</td>
<td>All</td>
<td>Table 3-8</td>
</tr>
<tr>
<td>Width of Shoulders</td>
<td>All</td>
<td>Table 3-8</td>
</tr>
<tr>
<td>Vertical Clearance, New Structures</td>
<td>All</td>
<td>5.0 m(^1),(^2)</td>
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<td>All</td>
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</tr>
<tr>
<td>Pavement Cross Slope</td>
<td>All</td>
<td>Chapter 2, Pavement Cross Slope</td>
</tr>
</tbody>
</table>

\(^1\) Exceptional cases near as practical to 16.5 ft. [5.0 m] but never less than 14.5 ft. [4.4 m].
\(^2\) Additional vertical clearance requirements will apply to roadways on the Texas Highway Freight Network (THFN) for projects Let on September 1, 2020 or later. See Ch. 3, Section 8 for specific requirements.

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

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>Design Speed (mph)</th>
<th>Minimum Width (^1,2)(ft.) for future ADT of:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>&lt; 400</td>
</tr>
<tr>
<td>Arterial Lanes</td>
<td></td>
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<tr>
<td>All</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Arterial Shoulders</td>
<td>4</td>
<td>4(^3), or 8(^4),/</td>
</tr>
</tbody>
</table>

\(^1\) \(^2\) Exceptional cases near as practical to 16.5 ft. [5.0 m] but never less than 14.5 ft. [4.4 m].
### Table 3-8: Width of Travel Lanes and Shoulders on Rural Two-lane Highways

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>Design Speed (mph)</th>
<th>Minimum Width (^1,2,3,4) (ft.) for future ADT of:</th>
<th></th>
<th></th>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>&lt; 400</td>
<td>400-1500</td>
<td>1500-2000</td>
<td>&gt; 2000</td>
<td></td>
</tr>
<tr>
<td>Collector</td>
<td>LANES (ft.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>10</td>
<td>10</td>
<td>11</td>
<td>12</td>
<td></td>
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<tr>
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<td>12</td>
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<td></td>
</tr>
<tr>
<td>SHOULDERS (ft.)</td>
<td>All</td>
<td>2(^5,7)</td>
<td>4(^5,7)</td>
<td>8(^5)</td>
<td>8 - 10(^5)</td>
<td></td>
</tr>
<tr>
<td>Local(^b)</td>
<td>LANES (ft)</td>
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<td></td>
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<tr>
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<td>10</td>
<td>10</td>
<td>11</td>
<td>12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SHOULDERS (ft.)</td>
<td>All</td>
<td>2(^1)</td>
<td>4(^1)</td>
<td>4(^1)</td>
<td>8</td>
<td></td>
</tr>
</tbody>
</table>

1 Minimum surfacing width is 24 ft. for all on-system state highway routes.
2 On high riprapped fills through reservoirs, a minimum of two 12 ft. lanes with 8 ft. shoulders should be provided for roadway sections. For arterials with 2,000 or more ADT in reservoir areas, two 12 ft. lanes with 10 ft. shoulders should be used.
3 On arterials, shoulders fully surfaced.
4 On collectors, use minimum 4 ft. shoulder width at locations where roadside barrier is utilized.
5 For collectors, shoulders fully surfaced for 1,500 or more ADT. Shoulder surfacing not required but desirable even if partial width for collectors with lower volumes and all local roads.
6 Applicable only to off-system routes that are not functionally classified at a higher classification.
7 A 5 ft. minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated except on off-system facilities with less than 400 ADT. See Ch. 2, Section 6 for additional information.
Table 3-8: Width of Travel Lanes and Shoulders on Rural Two-lane Highways

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>Design Speed (km/h)</th>
<th>Minimum Width 1,2(m) for future ADT of:</th>
<th>Metric</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>&lt; 400</td>
<td>400-1500</td>
</tr>
<tr>
<td>Arterial</td>
<td>LANES (m)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>SHOULders (m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>1.2 5</td>
<td>1.2 5 or 2.4 5</td>
</tr>
<tr>
<td>Collector</td>
<td>LANES (m)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>3.0</td>
<td>3.0</td>
<td>3.3</td>
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<td>3.3</td>
<td>3.3</td>
<td>3.6</td>
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<tr>
<td>110</td>
<td>3.3</td>
<td>3.3</td>
<td>3.6</td>
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<tr>
<td>120</td>
<td>3.3</td>
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<tr>
<td>130</td>
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<td>3.6</td>
</tr>
<tr>
<td>SHOULders (m)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>0.6 5</td>
<td>1.2 5</td>
</tr>
<tr>
<td>Local6</td>
<td>LANES (m)</td>
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<td></td>
</tr>
<tr>
<td>50</td>
<td>3.0</td>
<td>3.0</td>
<td>3.3</td>
</tr>
<tr>
<td>60</td>
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<tr>
<td>80</td>
<td>3.0</td>
<td>3.0</td>
<td>3.3</td>
</tr>
<tr>
<td>SHOULders (m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>0.6 5</td>
<td>1.2 5</td>
</tr>
</tbody>
</table>

1 Minimum surfacing width is 7.2 m for all on-system state highway routes.
2 On high riprapped fills through reservoirs, a minimum of two 3.6 m lanes with 2.4 m shoulders should be provided for roadway sections. For arterials with 2,000 or more ADT in reservoir areas, two 3.6 m lanes with 3.0 m shoulders should be used.
3 On arterials, shoulders fully surfaced.
4 On collectors, use minimum 1.2 m shoulder width at locations where roadside barrier is utilized.
5 For collectors, shoulders fully surfaced for 1,500 or more ADT. Shoulder surfacing not required but desirable even if partial width for collectors with lower volumes and all local roads.
6 Applicable only to off-system routes that are not functionally classified at a higher classification.
7 A 1.5 m minimum clear space for bicyclists should be provided on bridges being replaced or rehabilitated except on off-system facilities with less than 400 ADT. See Ch. 2, Section 6 for additional information.
Minimum width of new or widened structures should accommodate the approach roadway including shoulders.

See Table 3-9 for minimum structure widths that may remain in place.

Table 3-9: Minimum Structure Widths for Bridges to Remain in Place on Rural Two-lane Highways

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>Roadway Clear Width 1 (ft) for ADT of:</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 400</td>
<td>400-1500</td>
<td>1500-2000</td>
<td>&gt; 2000</td>
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<tr>
<td>Local</td>
<td>20</td>
<td>22</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td>Collector</td>
<td>22</td>
<td>22</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td>Arterial</td>
<td>Traveled Way + 6 ft</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Functional Class</th>
<th>Roadway Clear Width 1 (m) for ADT of:</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 400</td>
<td>400-1500</td>
<td>1500-2000</td>
<td>&gt; 2000</td>
</tr>
<tr>
<td>Local</td>
<td>6.0</td>
<td>6.6</td>
<td>7.2</td>
<td>8.4</td>
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<tr>
<td>Collector</td>
<td>6.6</td>
<td>6.6</td>
<td>7.2</td>
<td>8.4</td>
</tr>
<tr>
<td>Arterial</td>
<td>Traveled Way + 1.8 m</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 The Clear width on bridge structures without curbs is measured to the nominal face of rail. Reference the TxDOT Bridge Railing Manual and the Bridge Railing Standards for the nominal widths of specific rail types and additional guidance. For Bridges with curbs, the clear width is measured to the face of curb. The bridge clear width is considered to be at least the same as the approach roadway clear width. Approach roadway width includes shoulders.

Basic Design Features

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

- **Access Control**,
- **Transitions to Four-Lane Divided Highways**,
- **Passing Sight Distances**,
- **Speed Change Lanes**, and
- **Intersections**.
Access Control

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

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

Transitions to Four-Lane Divided Highways

Typical transitions from two-lane to four-lane divided highways are discussed in Transitions to Four-Lane Divided Highways, *Multi-Lane Rural Highways*, and illustrated in Figure 3-9: *Multi-Lane Rural Highway Intersection*.

Passing Sight Distances

Passing sight distance is the length of highway 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 conditions:

- 3.5 ft [1,080 mm] driver eye height,
- 3.5 ft [1,080 mm] object height, and
- 10 mph [15 km/h] speed differential between the passing vehicle and vehicle being passed.

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

Minimum passing sight distance values for design of two-lane highways are shown in Table 3-10. These distances are for design purposes only and should not be confused with other distances used as warrants for striping no-passing zones as shown in the *Texas Manual on Uniform Traffic Control Devices*. For the design of typical two-lane rural highways, except for level terrain, provision of near continuous passing sight distance (2.680 ft at 80 mph [815 m at 130 km/h]) is impractical. However, the designer should attempt to increase the length and frequency of passing sections where economically feasible.
Table 3-10: Passing Sight Distance

(US Customary)

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Minimum Passing Sight Distance for Design (ft.)</th>
<th>K-Value&lt;sup&gt;1&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>710</td>
<td>180</td>
</tr>
<tr>
<td>25</td>
<td>900</td>
<td>289</td>
</tr>
<tr>
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<td>1090</td>
<td>424</td>
</tr>
<tr>
<td>35</td>
<td>1280</td>
<td>585</td>
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<tr>
<td>40</td>
<td>1470</td>
<td>772</td>
</tr>
<tr>
<td>45</td>
<td>1625</td>
<td>943</td>
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<tr>
<td>50</td>
<td>1835</td>
<td>1203</td>
</tr>
<tr>
<td>55</td>
<td>1985</td>
<td>1407</td>
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<td>2135</td>
<td>1628</td>
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<td>65</td>
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<td>70</td>
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<td>75</td>
<td>2580</td>
<td>2377</td>
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<tr>
<td>80</td>
<td>2680</td>
<td>2565</td>
</tr>
</tbody>
</table>

(Metric)

<table>
<thead>
<tr>
<th>Design Speed (km/h)</th>
<th>Minimum Passing Sight Distance for Design (m)</th>
<th>K-Value&lt;sup&gt;1&lt;/sup&gt;</th>
</tr>
</thead>
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<td>200</td>
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<td>40</td>
<td>270</td>
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<td>70</td>
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<td>80</td>
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<td>730</td>
<td>617</td>
</tr>
<tr>
<td>120</td>
<td>775</td>
<td>695</td>
</tr>
<tr>
<td>130</td>
<td>815</td>
<td>769</td>
</tr>
</tbody>
</table>

<sup>1</sup>K = Length of Crest Vertical Curve ÷ Algebraic Difference in Grades

Speed Change Lanes

There are three kinds of speed change lanes: climbing lanes, left-turn lanes, and right-turn lanes.
**Climbing Lanes.** It is desirable to provide a climbing lane, as an extra lane on the upgrade side of a two-lane highway 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 [15 km/h] or greater speed reduction is expected for a typical heavy truck
- Level-of-service E or F exists on the upgrade
- A reduction of two or more levels of service is experienced when moving from the approach segment to the upgrade.

For low-volume roadways, only an occasional car is delayed, and a climbing lane may not be justified economically. For this reason, a climbing lane should only be considered on roadways with the following traffic conditions:

- Upgrade traffic flow rate in excess of 200 vehicles per hour or
- Upgrade truck flow rate in excess of 20 vehicles per hour.

The upgrade flow rate is predicted by multiplying the predicted or existing design hour volume by the directional distribution factor for the upgrade direction and dividing the result by the peak hour factor (see Traffic Characteristics, Chapter 2 and the Highway Capacity Manual for definitions of these terms). The upgrade truck flow rate is obtained by multiplying 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 desirably with a ratio of 25:1, but at least 150 ft [50 m] long.

Attention should also be given to the location of the climbing lane terminal. Ideally, the climbing lane should be extended to a point beyond the crest where a typical truck could attain a speed that is within 10 mph [15 km/h] 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 desirably with a ratio of 50:1.

For projects on new location or where an existing highway will be regraded, the economics of providing an improved grade line in lieu of providing climbing lanes should be investigated. Refer to Chapter 3 of AASHTO’s *A Policy on Geometric Design of Highways and Streets* for more information regarding the design of climbing lanes. Figure 3-5 shows cross sections for climbing lanes on rural highways.
Left-Turn Deceleration Lanes. Left-turn lanes on two-lane highways at intersecting crossroads generally are not economically justified. For certain moderate or high volume two-lane highways with heavy left-turn movements, however, left-turn lanes may be justified in view of reduced road user crash costs. Figure 3-6 provides recommendations for when left-turn lanes should be considered for a typical two-lane highway intersection.

Example: Traffic northbound on a highway has 350 vph with 10 percent left turns included. The southbound traffic volume is 200 vph. The design speed on the highway is 60 mph [100 km/h]. Beginning at the opposing volume (southbound in this case) of 200 vph, using the 10 percent left turn column and 60 mph [100 km/h] design speed section, a value of 330 vph advancing volume (northbound) is found in the table. Because the northbound volume of 350 vph exceeds the table value of 330 vph, a left turn lane should be considered at the intersection.
Lengths of left-turn deceleration lanes are provided in Table 3-11 for Two-Lane Highways and Table 3-13 for Multi-Lane Rural Highways.

Where used, left-turn lanes should be delineated with striping and pavement markers or jiggle bars. Passing should be restricted in advance of the intersection, and horizontal alignment shifts of the approaching travel lanes should be gradual. Figure 3-6 shows typical geometry for a rural two-lane highway with left-turn bays at a crossroad intersection.

![Figure 3-6. Typical Two-Lane Highway Intersection with Left-Turn Lanes.](image-url)
**Right-Turn Deceleration Lanes.** Shoulders 10 ft [3.0 m] 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 [3.0 m] with a 2 ft [0.6 m] surfaced shoulder. Where speed change lanes are used, they should be provided symmetrically along both sides of the highway for both directions of traffic, thus presenting drivers with a balanced section.

A deceleration-acceleration lane on one side of a two-lane highway, such as at a “tee” intersection, results in the appearance of a three-lane highway and may result in driver confusion. In this regard, right-turn speed change lanes are generally inappropriate for “tee” intersection design except where a four lane (2 through, 1 median left turn, 1 right acceleration/deceleration) section is provided.

---

**Table 3-11: Guide for Left-Turn Lanes on Two-Lane Highways**

<table>
<thead>
<tr>
<th>Opposing Volume (vph)</th>
<th>Advancing Volume (vph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5 % Left Turns</td>
</tr>
<tr>
<td>-----------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td><strong>40 mph [60 km/h] Design Speed</strong></td>
<td></td>
</tr>
<tr>
<td>800</td>
<td>330</td>
</tr>
<tr>
<td>600</td>
<td>410</td>
</tr>
<tr>
<td>400</td>
<td>510</td>
</tr>
<tr>
<td>200</td>
<td>640</td>
</tr>
<tr>
<td>100</td>
<td>720</td>
</tr>
<tr>
<td><strong>50 mph [80 km/h] Design Speed</strong></td>
<td></td>
</tr>
<tr>
<td>800</td>
<td>280</td>
</tr>
<tr>
<td>600</td>
<td>350</td>
</tr>
<tr>
<td>400</td>
<td>430</td>
</tr>
<tr>
<td>200</td>
<td>550</td>
</tr>
<tr>
<td>100</td>
<td>615</td>
</tr>
<tr>
<td><strong>60 mph [100 km/h] Design Speed</strong></td>
<td></td>
</tr>
<tr>
<td>800</td>
<td>230</td>
</tr>
<tr>
<td>600</td>
<td>290</td>
</tr>
<tr>
<td>400</td>
<td>365</td>
</tr>
<tr>
<td>200</td>
<td>450</td>
</tr>
<tr>
<td>100</td>
<td>505</td>
</tr>
</tbody>
</table>
Section 2, Figure 3-4 shows the lengths for right-turn deceleration lanes.

The length of a right-turn deceleration lane is the same as that for a left-turn lane (see Table 3-13). Right turn lanes shorter than the lengths given in Table 3-13 may be acceptable on some low volume rural highways.

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

**Intersections**

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

Desirably, the roadways should intersect at approximately right angles, and should not intersect less than 75 degrees. Where crossroad skew is flatter than 75 degrees to the highway, the crossroad should be re-aligned to provide for a near perpendicular crossing. The higher the functional classification, the closer to right-angle the crossroad intersection should be.

Minimum Designs for Truck and Bus Turns in Chapter 7 provides information regarding the accommodation of various types of truck class vehicles in intersection design. Further information on intersection design may also be found in AASHTO’s A Policy on Geometric Design of Highways and Streets.
Section 5 — Multi-Lane Rural Highways

Overview

This section includes guidelines on geometric features for multilane rural highways. The guidelines are outlined in Table 3-12, and Figure 3-7 and Figure 3-8. The guidelines apply for all functional classes of roadways.

Table 3-12: Design Criteria For Multilane Rural Highways
(Non-controlled Access) (All Functional Classes)

<table>
<thead>
<tr>
<th>Type of Facility</th>
<th>Six-Lane Divided</th>
<th>Four-Lane Divided</th>
<th>Four-Lane Undivided</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed (Arterials)(^1) (mph)</td>
<td>Min.</td>
<td>Min.</td>
<td>Min.</td>
</tr>
<tr>
<td>Level</td>
<td>70(^2)</td>
<td>70(^2)</td>
<td>70(^2)</td>
</tr>
<tr>
<td>Rolling</td>
<td>60(^3)</td>
<td>60(^3)</td>
<td>60(^3)</td>
</tr>
<tr>
<td>Vertical Clearance, New Structures</td>
<td>16.5(^5,6)</td>
<td>16.5(^5,6)</td>
<td>16.5(^5,6)</td>
</tr>
<tr>
<td>Lane Width (ft)</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Median Width (ft)</td>
<td>Surfaced</td>
<td>16</td>
<td>4</td>
</tr>
<tr>
<td>-</td>
<td>Depressed</td>
<td>76</td>
<td>48</td>
</tr>
<tr>
<td>Shoulder Outside (ft)</td>
<td>10</td>
<td>8(^4)</td>
<td>10</td>
</tr>
<tr>
<td>Shoulder Inside (ft) for Depressed Medians</td>
<td>10</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Min. Structure Widths for Bridges to Remain in place (ft)</td>
<td>Depressed Median</td>
<td>--</td>
<td>42(^7)</td>
</tr>
</tbody>
</table>
For multilane collectors, minimum design speed values are 10 mph less than tabulated.

2 60 mph acceptable when conditions warrant and are documented through a design exception.

3 50 mph acceptable when conditions warrant and are documented through a design exception.

4 Applies to collector roads only. On four-lane undivided 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.

5 Additional vertical clearance requirements will apply to roadways on the Texas Highway Freight Network (THFN) for projects Let on September 1, 2020 or later. See Ch. 3, Section 8 for specific requirements.

6 Exceptional cases near as practical to 16.5 ft. but never less than 14.5 ft.

7 The Clear width on bridge structures without curbs is measured to the nominal face of rail. Reference the TxDOT Bridge Railing Manual and Bridge Railing Standards for the nominal widths of specific rail types and additional guidance. For Bridges with curbs, the clear width is measured to the face of curb. The Bridge Clear width is considered to be at least the same as the approach roadway clear width. The approach roadway width includes shoulders.

### Table 3-12: Design Criteria For Multilane Rural Highways (Non-controlled Access) (All Functional Classes)

<table>
<thead>
<tr>
<th>Type of Facility</th>
<th>Six-Lane Divided</th>
<th>Four-Lane Divided</th>
<th>Four-Lane Undivided</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed (Arterials)(^1) (km/h)</td>
<td>Min.</td>
<td>Min.</td>
<td>Min.</td>
</tr>
<tr>
<td>Flat</td>
<td>110(^2)</td>
<td>110(^2)</td>
<td>110(^2)</td>
</tr>
<tr>
<td>Rolling</td>
<td>100(^3)</td>
<td>100(^3)</td>
<td>100(^3)</td>
</tr>
<tr>
<td>Vertical Clearance, New Structures</td>
<td>(5^{5,6})</td>
<td>(5^{5,6})</td>
<td>(5^{5,6})</td>
</tr>
<tr>
<td>Lane Width (m)</td>
<td>3.6</td>
<td>3.6</td>
<td>3.6</td>
</tr>
<tr>
<td>Median Width (m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surfaced</td>
<td>4.8</td>
<td>1.2</td>
<td>4.8</td>
</tr>
<tr>
<td>Depressed</td>
<td>22.8</td>
<td>14.4</td>
<td>22.8</td>
</tr>
<tr>
<td>Shoulder Outside (m)</td>
<td>3.0</td>
<td>2.4(^4)</td>
<td>3.0</td>
</tr>
</tbody>
</table>
### Table 3-12: Design Criteria For Multilane Rural Highways
(Non-controlled Access) (All Functional Classes)

<table>
<thead>
<tr>
<th>Shoulder Inside (m) for Depressed Medians</th>
<th>3.0</th>
<th>1.2</th>
<th>1.2</th>
<th>1.2</th>
<th>Not applicable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min. Structure Widths for Bridges to Remain in place (m)</td>
<td>Depressed Median</td>
<td>--</td>
<td>12.6</td>
<td>--</td>
<td>9.0</td>
</tr>
</tbody>
</table>

1. For multilane collectors, minimum design speed values are 20 km/h less than tabulated.
2. 100 km/h acceptable when conditions warrant and are documented through a design exception.
3. 80 km/h acceptable when conditions warrant and are documented through a design exception.
4. Applies to collector roads only. On four-lane undivided highways, outside surfaced shoulder width may be decreased to 1.2 m where flat (1V:10H), sodded front slopes are provided for a minimum distance of 1.2 m from the shoulder edge.
5. Additional vertical clearance requirements will apply to roadways on the Texas Highway Freight Network (THFN) for projects Let on September 1, 2020 or later. See Ch. 3, Section 8 for specific requirements.
6. Exceptional cases near as practical to 5.0 m but never less than 4.4 m.
7. The Clear width on bridge structures without curbs is measured to the nominal face of rail. Reference the TxDOT Bridge Railing Manual and Bridge Railing Standards for the nominal widths of specific rail types and additional guidance. For Bridges with curbs, the clear width is measured to the face of curb. The Bridge Clear width is considered to be at least the same as the approach roadway clear width. The approach roadway width includes shoulders.
TYPICAL MULTILANE SECTION (Undivided Highway)

Notes:

1. Slope may be exceeded in rock cuts, for restricted right of way or deep cut conditions, or where ditch is not within horizontal clearances.

2. See Table 2-11.

3. See Chapter 2, Slopes and Ditches.

4. See Table 2-14.

CROSS SECTIONS FOR ARTERIAL AND COLLECTOR MULTI-LANE UNDIVIDED RURAL HIGHWAYS (US CUSTOMARY)

Figure 3-7. (US). Cross Sections For Arterial and Collector Multi-Lane Undivided Rural Highways.
References to other applicable criteria are as follows:

- Minimum Horizontal Radius: Table 2-3: Horizontal Curvature of High-Speed Highways and Connecting Roadways with Superelevation and Figure 2-2, Determination of Length of Superelevation Transition.
- Maximum Gradient: Table 2-11: Maximum Grades
- Fill Slope Rates: Table 8-11: Earth Fill Slope Rates.

Level of Service

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

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

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

**Basic Design Criteria**

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

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

**Access Control**

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

For multilane highways constructed in developed (or expected to be developed) areas, such as bypasses in close proximity to urban areas, it may be desirable to control access to the main lanes by either purchasing access rights as part of the right-of-way acquisition or by design (i.e., provision of frontage roads). Where desired, control of access by design may be provided either solely in the interchange areas or continuously throughout a section of highway, depending on traffic volumes, the degree of roadside development, availability of right-of-way, economic conditions, etc.

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

**Medians**

The width of the median is the distance between the inside edges of the travel lanes. Insofar as practical, wide (desirably 76 ft [22.8 m]) medians should be used to provide sufficient storage space for tractor-trailer combination vehicles at median openings, reduce headlight glare, provide a pleasing appearance, and reduce the chances of head-on collisions. However, in areas that are likely
to become suburban or urban in nature, medians wider than 60 ft [18 m] should be avoided at inter-
sections except where necessary to accommodate turning and crossing maneuvers by larger
vehicles. Wide medians may be a disadvantage when signalization is required at intersections. The
increased time for vehicles to cross the median can lead to inefficient signal operation.

**Four-Lane Undivided Highways.** Improvement of an existing two-lane highway to a four-lane
highway facility preferably should include a median, but four-lane undivided highways may
be considered for existing two-lane highways to improve passing opportunities and traffic
operations in certain instances such as rolling terrain, or where restricted right-of-
way conditions and moderate traffic volumes dictate. Table 3-12: Design Criteria For Multilane
Rural Highways (Non-controlled Access) (All Functional Classes) and Figure 3-7 include the gen-
eral geometric features for four-lane undivided highways.

**Surfaced Medians.** Surfaced medians of 4 ft to 16 ft [1.2 m to 4.8 m] are classified as narrow
medians and are used in restricted conditions. Medians 4 ft [1.2 m] wide provide little separation of
opposing traffic and a minimal refuge area for pedestrians. Surfaced medians of 14 ft to 16 ft [4.2 m
to 4.8 m] offer space for use by exiting traffic turning left, but do not offer protection for crossing
vehicles. Surfaced median designs are most appropriate in areas with roadside development.

**Wide Medians.** Medians 76 ft [22.8 m] wide significantly reduce headlight glare, are pleasing in
appearance, reduce the chances of head-on collisions, and provide a sheltered storage area for
crossing vehicles, including tractor-trailer combinations. Wide medians should generally be used
whenever feasible but median widths greater than 60 ft [18 m] have been found to be undesirable
for intersections that are signalized or may be signalized in the design life of the project.

**Median Openings.** Median openings at close intervals on divided highways can cause interference
between high-speed through-traffic and turning vehicles. The frequency of median openings varies
with topographic restrictions and local requirements; however, as a general rule the minimum spac-
ing should not be less than one-quarter mile [400 m] in rural areas. Spacing often is selected to
provide openings at all public roads and at major traffic generators such as industrial sites or shop-
ing centers. Additional openings should be provided so as not to surpass a maximum one-half
mile [800 m] spacing.

Left-turn lanes should be provided at all median openings. At intersections with highways or other
major public roads, turn lanes for right-turning vehicles entering and exiting the highway are usu-
ally provided, as shown in Figure 3-9. For divided highways with independent main lane
alignment, particular care should be exercised at median openings to provide a satisfactory profile
along the crossover with flat, platform approaches to the main lanes.
Median opening width should in no case be less than 40 ft [12 m] or less than crossroad pavement width plus 8 ft [2.4 m]. Turning templates for a selected control radius and design vehicle are often used as the basis for minimum design of median openings, particularly for multilane crossroads and skewed intersections. See Minimum Designs for Truck and Bus Turns for additional information.

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

**Table 3-13: Lengths of Median Turn Lanes Multilane Rural Highways**

<table>
<thead>
<tr>
<th>Mainlane Design Speed (mph)</th>
<th>Taper Length (ft)</th>
<th>Deceleration Length (ft)</th>
<th>Design Turning ADT (vpd)</th>
<th>Minimum Storage Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>50</td>
<td>160</td>
<td>150</td>
<td>50</td>
</tr>
<tr>
<td>35</td>
<td>50</td>
<td>215</td>
<td>300</td>
<td>100</td>
</tr>
<tr>
<td>40</td>
<td>50</td>
<td>275</td>
<td>500</td>
<td>175</td>
</tr>
<tr>
<td>45</td>
<td>100</td>
<td>345</td>
<td>750</td>
<td>250</td>
</tr>
<tr>
<td>50</td>
<td>100</td>
<td>425</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>55</td>
<td>100</td>
<td>510</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>60</td>
<td>150</td>
<td>615</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>65</td>
<td>150</td>
<td>715</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>70</td>
<td>150</td>
<td>830</td>
<td>--</td>
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</tr>
<tr>
<td>75</td>
<td>150</td>
<td>950</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>80</td>
<td>150</td>
<td>1075</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mainlane Design Speed (km/h)</th>
<th>Taper Length (m)</th>
<th>Deceleration Length (m)</th>
<th>Design Turning ADT (vpd)</th>
<th>Minimum Storage Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>15</td>
<td>50</td>
<td>150</td>
<td>15</td>
</tr>
<tr>
<td>60</td>
<td>15</td>
<td>65</td>
<td>300</td>
<td>30</td>
</tr>
<tr>
<td>70</td>
<td>30</td>
<td>85</td>
<td>500</td>
<td>50</td>
</tr>
<tr>
<td>80</td>
<td>30</td>
<td>105</td>
<td>750</td>
<td>75</td>
</tr>
<tr>
<td>90</td>
<td>30</td>
<td>130</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>100</td>
<td>45</td>
<td>200</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>110</td>
<td>45</td>
<td>240</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>120</td>
<td>45</td>
<td>290</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>130</td>
<td>45</td>
<td>330</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

1. For low volume median openings, such as those serving private drives or U-turns, a taper length of 100 ft [30 m] may be used regardless of mainlane design speed.

2. Deceleration length assumes that moderate deceleration will occur in the through traffic lane and the vehicle entering the left-turn lane will clear the through traffic lane at a speed of 10 mph (15 km/h) slower than through traffic. Where providing this deceleration length is impractical, it may be acceptable to allow turning vehicles to decelerate more than 10 mph (15km/h) before clearing the through traffic lane.

**Right Turn Deceleration Lane.** Right (12 ft [3.6 m] lane with 4 ft [1.2 m] adjacent shoulders) turn lanes provide deceleration or acceleration area for right-turning vehicles. The deceleration length and taper lengths for right turn lanes are the same as for left-turn lanes (See Table 3-13). Adjustment factors for grade effects are shown in Table 3-14.
**Acceleration Lanes.** Acceleration lanes for right-turning and/or left-turning vehicles may be desirable on multi-lane rural highways. Acceleration distances and taper lengths are provided in Figure 3-10. Adjustments for grade are given in Table 3-14.

![Figure 3-10. (US). Lengths of Right-Turn Acceleration Lanes.](Figure_3-10.png)

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

<table>
<thead>
<tr>
<th>Highway Design Speed (mph)</th>
<th>Minimum Length of Taper T (ft)</th>
<th>Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)</th>
<th>Stop Condition</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deceleration Lanes</td>
<td></td>
<td></td>
<td></td>
<td>0</td>
<td>14</td>
<td>18</td>
<td>22</td>
<td>26</td>
<td>30</td>
<td>35</td>
<td>40</td>
</tr>
<tr>
<td>Design Speed of Roadway</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(mph)</td>
<td></td>
<td></td>
<td></td>
<td>30</td>
<td>150</td>
<td>180</td>
<td>140</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(US Customary)</td>
<td></td>
<td></td>
<td></td>
<td>35</td>
<td>165</td>
<td>280</td>
<td>220</td>
<td>160</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(ft)</td>
<td></td>
<td></td>
<td></td>
<td>40</td>
<td>180</td>
<td>360</td>
<td>300</td>
<td>270</td>
<td>210</td>
<td>120</td>
<td>-</td>
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<tr>
<td>(US Customary)</td>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>200</td>
<td>560</td>
<td>490</td>
<td>440</td>
<td>380</td>
<td>280</td>
<td>160</td>
</tr>
<tr>
<td>(ft)</td>
<td></td>
<td></td>
<td></td>
<td>50</td>
<td>230</td>
<td>720</td>
<td>660</td>
<td>610</td>
<td>550</td>
<td>450</td>
<td>350</td>
</tr>
<tr>
<td>(US Customary)</td>
<td></td>
<td></td>
<td></td>
<td>55</td>
<td>250</td>
<td>960</td>
<td>900</td>
<td>810</td>
<td>780</td>
<td>670</td>
<td>550</td>
</tr>
<tr>
<td>(ft)</td>
<td></td>
<td></td>
<td></td>
<td>60</td>
<td>265</td>
<td>1200</td>
<td>1140</td>
<td>1100</td>
<td>1020</td>
<td>910</td>
<td>800</td>
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<td></td>
<td></td>
<td></td>
<td>65</td>
<td>285</td>
<td>1410</td>
<td>1350</td>
<td>1310</td>
<td>1220</td>
<td>1120</td>
<td>1000</td>
</tr>
<tr>
<td>(ft)</td>
<td></td>
<td></td>
<td></td>
<td>70</td>
<td>300</td>
<td>1620</td>
<td>1560</td>
<td>1520</td>
<td>1420</td>
<td>1350</td>
<td>1230</td>
</tr>
<tr>
<td>(US Customary)</td>
<td></td>
<td></td>
<td></td>
<td>75</td>
<td>330</td>
<td>1790</td>
<td>1730</td>
<td>1630</td>
<td>1580</td>
<td>1510</td>
<td>1420</td>
</tr>
</tbody>
</table>

*Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,500 ft.*
### Table 3-14: Speed Change Lane Adjustment Factors as a Function of a Grade

#### (US Customary)

<table>
<thead>
<tr>
<th>Speed of Roadway (mph)</th>
<th>3 to 4% Upgrade</th>
<th>3 to 4% Downgrade</th>
<th>5 to 6% Upgrade</th>
<th>5 to 6% Downgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>****</td>
<td>****</td>
<td>****</td>
<td>****</td>
</tr>
<tr>
<td>40</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>45</td>
<td>1.3</td>
<td>1.35</td>
<td>1.35</td>
<td>****</td>
</tr>
<tr>
<td>50</td>
<td>1.3</td>
<td>1.35</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>55</td>
<td>1.35</td>
<td>1.4</td>
<td>1.45</td>
<td>1.45</td>
</tr>
<tr>
<td>60</td>
<td>1.4</td>
<td>1.45</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>65</td>
<td>1.45</td>
<td>1.5</td>
<td>1.55</td>
<td>1.6</td>
</tr>
<tr>
<td>70</td>
<td>1.5</td>
<td>1.55</td>
<td>1.65</td>
<td>1.75</td>
</tr>
<tr>
<td>75</td>
<td>1.55</td>
<td>1.6</td>
<td>1.65</td>
<td>1.75</td>
</tr>
<tr>
<td>80</td>
<td>1.6</td>
<td>1.65</td>
<td>1.75</td>
<td>1.85</td>
</tr>
</tbody>
</table>

#### 5 to 6% Upgrade

<table>
<thead>
<tr>
<th>Speed of Roadway (mph)</th>
<th>5 to 6% Downgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>****</td>
</tr>
<tr>
<td>35</td>
<td>****</td>
</tr>
<tr>
<td>40</td>
<td>1.5</td>
</tr>
<tr>
<td>45</td>
<td>1.55</td>
</tr>
<tr>
<td>50</td>
<td>1.6</td>
</tr>
<tr>
<td>55</td>
<td>1.65</td>
</tr>
<tr>
<td>60</td>
<td>1.7</td>
</tr>
<tr>
<td>65</td>
<td>1.85</td>
</tr>
<tr>
<td>70</td>
<td>2.0</td>
</tr>
<tr>
<td>75</td>
<td>2.2</td>
</tr>
<tr>
<td>80</td>
<td>2.4</td>
</tr>
</tbody>
</table>

*Ratio in this table multiplied by length of deceleration or acceleration distances in Table 3-13 and Figure 3-10, gives length of deceleration/acceleration distance on grade.

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

#### (Metric)

<table>
<thead>
<tr>
<th>Design Speed of Roadway (km/h)</th>
<th>Ration of Length on Grade to Length on Level*</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 to 4% Upgrade</td>
<td>5 to 6% Upgrade</td>
</tr>
<tr>
<td>40</td>
<td>50</td>
</tr>
<tr>
<td>60</td>
<td>70</td>
</tr>
<tr>
<td>80</td>
<td>All Speeds</td>
</tr>
<tr>
<td>0.9</td>
<td>1.2</td>
</tr>
<tr>
<td>0.8</td>
<td>1.35</td>
</tr>
</tbody>
</table>

*Ratio in this table multiplied by length of deceleration or acceleration distances in Table 3-13 and Figure 3-10, gives length of deceleration/acceleration distance on grade.
Design Criteria

Section 5 — Multi-Lane Rural Highways

Travel Lanes and Shoulders

**Travel Lanes.** Travel lanes should be 12 ft [3.6 m] minimum width on rural multilane highways. *The Highway Capacity Manual* should be consulted to determine the number of lanes to be used in the design.

**Shoulders.** Shoulders should be provided with widths as shown in Table 3-12: Design Criteria For Multilane Rural Highways (Non-controlled Access) (All Functional Classes).

Intersections

In the design of intersections, careful consideration should be given to the appearance of the intersection from the driver’s perspective. In this regard, design should be rather simple to avoid driver confusion. In addition, adequate sight distance should be provided throughout, especially in maneuver or conflict areas. See *Stopping Sight Distance* in Chapter 2 for further information regarding sight distance. For guidance on Alternative Intersections see Appendix E.

| Table 3-14: Speed Change Lane Adjustment Factors as a Function of a Grade (Metric) |
|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
|                | 50              | 60              | 70              | 80              | 90              | 100             |
|                | 1.3             | 1.4             | 1.4             | 1.5             | 1.5             | 1.6             |
|                | 1.3             | 1.5             | 1.5             | 1.5             | 1.5             | 1.6             |
|                | 1.4             | 1.5             | 1.5             | 1.5             | 1.5             | 1.6             |
|                | 1.4             | 1.5             | 1.5             | 1.5             | 1.5             | 1.6             |
|                | 1.5             | 1.6             | 1.7             | 1.7             | 1.7             | 1.8             |
|                | 1.5             | 1.6             | 1.7             | 1.7             | 1.7             | 1.8             |
|                | 1.5             | 1.6             | 1.7             | 1.7             | 1.7             | 1.8             |
|                |                |                |                |                |                |                |
| 5 to 6% Upgrade| 5 to 6% Downgrade|
| 50              |                |                |                |                |                |                |
| 60              | 1.5            | 1.5            |                |                |                |                |
| 70              | 1.5            | 1.6            | 1.7            |                |                |                |
| 80              | 1.5            | 1.7            | 1.9            | 1.8            |                |                |
| 90              | 1.6            | 1.8            | 2.0            | 2.1            | 2.2            |                |
| 100             | 1.7            | 1.9            | 2.2            | 2.4            | 2.5            | 0.5            |
| 110             | 2.0            | 2.2            | 2.6            | 2.8            | 3.0            | 0.5            |
| 120             | 2.3            | 2.5            | 3.0            | 3.2            | 3.5            | 0.5            |
| 130             | 2.6            | 2.8            | 3.4            | 3.6            | 4.0            | 0.5            |

*Ratio in this table multiplied by length of deceleration or acceleration distances in Table 3-13 and Figure 3-10, gives length of deceleration/acceleration distance on grade.*
Right angle crossings are preferred to skewed crossings, and where skew angles exceed 60 degrees, alignment modifications are generally necessary. Turn Lanes may be provided in accordance with previous discussions.

Chapter 7, Minimum Designs for Truck and Bus Turns provides information regarding the accommodation of various types of truck class vehicles in intersection design. AASHTO’s A Policy on Geometric Design of Highways and Streets should be consulted for further information on intersection design and intersection sight distance.

Intersections formed at by-pass and existing route junctions should be designed so as not to mislead drivers. Treatment of an old-new route connection is illustrated in Figure 3-11.

For intersections with narrow, depressed median sections, it may be necessary to affect superelevation across the entire cross section to provide for safer operation at median openings.

For more information on intersection design, see Stopping Sight Distance in Chapter 2.

For more information on border areas, see Borders.

Figure 3-11. Treatment of Old-New Route Connection at Point Where Relocation Begins.
Transitions to Four-Lane Divided Highways

Typical transitions from a two-lane to a four-lane divided highway are shown in Figure 3-12. Transition geometrics should meet the design criteria 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 [400 m] of the transition area. Transition areas should be located so that obstructions such as restrictive width bridges or underpasses or other fixed objects are not within the no-passing zone of the two-lane highway approach.

Figure 3-12. (US). Typical Transitions from Two-Lane to Four-Lane Divided Highways.

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

The Federal Highway Administration will allow the existing alignments to remain in place when existing two-lane roadways are converted to four-lane divided facilities. 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 in the horizontal or vertical alignment of the existing road. Other features such as signing, roadside hardware, safety end treatments, etc., should meet current standards.
Existing structures with substandard width on the existing lanes may remain if that width meets minimum rehabilitation (3R) requirements for multi-lane facilities.

A crash analysis of the existing two-lane roadway should be conducted. Any specific areas involving high crash frequencies will be reviewed and corrective measures taken where appropriate.

**Grade Separations and Interchanges**

Grade separations or interchanges on multilane rural highways may be provided at high-volume highway or railroad crossings, or to increase safety at crash-prone crossings.

Further information on grade separations and interchanges may be found in Chapter 3, Freeways and Chapter 10 of AASHTO’s *A Policy on Geometric Design of Highways and Streets*. 
Section 6 — Freeways

Overview

A freeway is defined as a controlled access multilane divided facility. Freeways are functionally classified as arterials but have unique design characteristics that set them apart from non-access controlled arterials. This section discusses the features and design criteria for freeways and includes the following subsections:

- Basic Design Criteria
- Access Control
- Mainlane
- **Vertical and Clear Zones at Structures**
- Frontage Roads
- Interchanges
- Ramps

Basic Design Criteria

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

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td></td>
</tr>
<tr>
<td>Clear Zone</td>
<td>Table 2-12</td>
</tr>
<tr>
<td>Grades</td>
<td>Table 2-11</td>
</tr>
<tr>
<td>Minimum Horizontal Radius</td>
<td>Table 2-3 and 2-4</td>
</tr>
<tr>
<td>Superelevation</td>
<td>Tables 2-6, 2-7 and 2-8</td>
</tr>
<tr>
<td>Vertical Curvature</td>
<td>Figures 2-6 and 2-7</td>
</tr>
<tr>
<td>Pavement Cross Slope</td>
<td>Chapter 2, Pavement Cross Slope</td>
</tr>
<tr>
<td>Lane and Shoulder Widths</td>
<td>Table 3-18</td>
</tr>
<tr>
<td>Freeways</td>
<td></td>
</tr>
<tr>
<td>Design Speed Mainlanes (urban and rural)</td>
<td>Table 3-17</td>
</tr>
<tr>
<td>Design Speed Frontage Roads (urban and rural)</td>
<td>Chapter 3, Frontage Roads</td>
</tr>
<tr>
<td>Capacity and LOS Analysis</td>
<td>Highway Capacity Manual</td>
</tr>
</tbody>
</table>
Access Control

This subsection discusses access control and includes the following topics:

- **General**
- **Mainlane Access**
- **Frontage Road Access**
- **Driveways and Side Streets**
- **Methods**

**General**

The entire Interstate Highway System and portions of the State Highway System have been designated by the Commission as Controlled Access Highways, thereby making it necessary along certain sections of said highways to either limit or completely deny the abutting owner’s access rights, which include the right of ingress and egress and the right of direct access to and from said owner’s abutting property to said highway facility. Such access may be controlled under the State’s Police Power, which is an inherent right of sovereignty. However, the existing right of access to an existing public way is an increment of ownership and a part of the bundle of rights vested in the owner of abutting property. It is a legal right, and though such right may be limited or completely denied under the State’s Police Power, the owner is entitled to be paid whatever damages may be suffered by reason of the loss of such access.

The abutting owners are denied access to any controlled access highway on new location, unless there is a specific grant of access, and no damages may be claimed for the denial of access to the new facility; the theory being that the owner cannot be damaged by the loss of something which the owner never had.

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

If an existing road is converted into a controlled access facility and no part of the abutting owner’s property is taken for right of way, but access is to be denied to the controlled access facility, and by reason of such denial of access it is found that such owner will suffer damages measured by the
diminution of the market value of said abutting land, said owner should be requested to release and relinquish said access rights for consideration equal to the State’s approved value for such damages. If the owner is not willing to negotiate on these terms, then the access right may be acquired through eminent domain proceedings. In some instances, the State’s appraisal and approved value may indicate that there is no diminution in value by reason of the access denial, and in those cases the abutting owner should be requested to release and relinquish access rights for no cash consideration. If the owner refuses to do so, then the access rights should be acquired through eminent domain proceedings with the State testifying to a zero value for such rights.

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
- Through construction of frontage roads permitting access to the mainlanes only at selected ramps.

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

Frontage Road Access

In the case where frontage roads are provided, access should be controlled for operational purposes at ramp junctions with frontage roads through access restrictions or the use of the State’s police powers to control driveway location and design. Figures 3-13 and 3-14 show recommended access control strategies for planned exit and entrance ramps, respectively, and should be used where practical.
Figure 3-13. Recommended Access Control at Exit Ramp Junction with Frontage Road.
Driveways and Side Streets

The placement of streets and driveways in the vicinity of freeway ramp/frontage road intersections should be carefully considered and permitted only after local traffic operations are considered. Information on the driveway clearance from the cross street intersection is contained in the TxDOT Access Management Manual and should be considered in the locating of any driveways on projects involving the construction or reconstruction of ramps and/or frontage roads.

Figure 3-14. Recommended Access Control at Entrance Ramp Junction with Frontage Road.
Table 3-16 shows the 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.

<table>
<thead>
<tr>
<th>Total Volume (Frontage rd + Ramp) (vph)</th>
<th>Driveway or Side Street Volume (vph)</th>
<th>Spacing (ft [m])</th>
</tr>
</thead>
<tbody>
<tr>
<td>-- -- Number of Weaving Lanes</td>
<td>2 3 4</td>
<td></td>
</tr>
<tr>
<td>&lt; 2500</td>
<td>460 [140] 460 [140] 560 [170]</td>
<td></td>
</tr>
<tr>
<td>-- &gt; 250</td>
<td>520 [160] 460 [140] 560 [170]</td>
<td></td>
</tr>
<tr>
<td>-- &gt; 750</td>
<td>790 [240] 460 [140] 560 [170]</td>
<td></td>
</tr>
<tr>
<td>-- &gt; 1000</td>
<td>1000 [300] 460 [140] 560 [170]</td>
<td></td>
</tr>
<tr>
<td>&gt; 2500</td>
<td>920 [280] 460 [140] 560 [170]</td>
<td></td>
</tr>
<tr>
<td>-- &gt; 250</td>
<td>950 [290] 460 [140] 560 [170]</td>
<td></td>
</tr>
<tr>
<td>-- &gt; 750</td>
<td>1000 [300] 600 [180] 690 [210]</td>
<td></td>
</tr>
<tr>
<td>-- &gt; 1000</td>
<td>1000 [300] 1000 [300] 1000 [300]</td>
<td></td>
</tr>
</tbody>
</table>

Driveway or side street access on the frontage road in close downstream proximity to exit ramp terminals increases the weaving that occurs on the frontage road and may lead to operational problems. For this reason, it is important to maintain appropriate separation between the intersection of the exit ramp and frontage road travel lanes, and downstream driveways or side streets where practical.

It is recognized that there are occasions when meeting these exit ramp separation distance values may not be possible due to the nature of the existing development, such as a high number of closely spaced driveways and/or side streets especially when in combination with closely spaced interchanges. In these cases, at least 250 ft [75 m] of separation should be provided between the intersection of the exit ramp and frontage road travel lanes and the downstream driveway or side street. Since the use of only 250 ft [75 m] of separation distance may negatively impact the operation of the frontage road, exit ramp, driveway and/or side street traffic, careful consideration should be given to its use. When the 250 ft [75 m] separation distance cannot be obtained, consideration should be given to channelization methods that would restrict access to driveways within this 250 ft [75 m] distance. Refer to the Texas MUTCD for specific types of channelization.

There will be similar occasions when meeting the entrance ramp separation distance values may not be possible due to the same existing development conditions associated with exit ramps. In these cases, at least 100 ft [30 m] 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.
Since the use of only 100 ft [30 m] of entrance ramp separation distance may also negatively impact the operation of the frontage road, entrance ramp, driveway, and/or side street traffic, careful consideration should be given to its use. As with exit ramps, when the 100 ft [30 m] entrance ramp separation distance cannot be obtained, consideration should be given to channelization methods that would restrict access to driveways within this 100 ft [30 m] distance. Refer to the Texas MUTCD for specific types of channelization.

**Relocating Driveways.** On reconstruction projects, it may be necessary to close or relocate driveways in order to meet these guidelines. However, 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 through a traffic operations and safety evaluation.

**Ramp Location.** In the preparation of schematic drawings, care should be exercised to develop design in sufficient detail to accurately tie down the locations of ramp junctions with frontage roads and thus the location of access control limits. The control of access lines shall be shown on the schematic drawings at entrance and exit ramp junctions with frontage roads as shown in Figures 3-13 and 3-14. These drawings are often displayed at meetings and hearings and further become the basis for right-of-way instruments or, in some cases, the Department's regulation of driveway location for that project.

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

**Methods**

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

- **Designation** (Transportation Code §203.031 and access restrictions)
- **Design** (continuous frontage road and State’s police power)

**Control of Access by Designation**

When the Texas Transportation Commission designates a freeway to be developed as a controlled access facility under Transportation Code §203.031, the State is empowered to control access through access restrictions. All Interstate Highways are designated as controlled access and certain other routes have been or may be designated. These designated freeways may or may not have frontage roads, whichever arrangement is determined to be appropriate as discussed in Planning Development of freeways by designation, rather than solely by design, is the preferred design approach especially for all new location freeways.
Under Transportation Code §203.031, Not Along an Existing Public Road. Whenever designated controlled access freeways include frontage roads and the planned location is not along an existing public road, preferably access should be controlled through access restrictions at ramp junctions with frontage roads as shown on Figure 3-13 and Figure 3-14.

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

Under Transportation Code §203.031, Along an Existing Public Road. Whenever a designated controlled access freeway is to be provided along the location of an existing public road, generally (subject to discussion in Planning) frontage roads are provided to retain or restore existing access.

Frontage road access should be controlled by imposing access restrictions in accordance with Figure 3-13 and Figure 3-14 whenever all of the following conditions prevail:

- Right-of-way is being obtained from the abutting property owner(s).
- A landlocked condition does not result.
- Recommended control of access as shown in Figure 3-13 and Figure 3-14.

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

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

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

Control of Access by Design

If an existing highway is to be developed as a controlled access facility solely by design (not designated by the Transportation Commission), TxDOT is not empowered to purchase access rights but 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 locations such as ramp junctions with frontage roads.

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

- Reasonable access is provided.
- Land locking of an abutting property does not result.
The TxDOT Access Management Manual governs design and location of driveways.

Whenever new or relocated ramps are to be provided along existing freeways, the design philosophy shown in Frontage Roads applies. Access should therefore be controlled at frontage road junctions through access restriction as illustrated in Figures 3-13 and 3-14.

Whenever access is to be controlled solely by provision of frontage roads, departmental 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, it is preferred over controlling access solely by design.

Mainlanes

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

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

Design Speed

The design speed of urban freeways should reflect the desired operating conditions during non-peak hours. The design speed should not exceed the limits of prudent construction, right-of-way, and socioeconomic costs because a large proportion of vehicles are accommodated during periods of peak flows when lower speeds are tolerable. Design speeds for rural freeways should be high, providing a design speed that is consistent with the overall quality and safety of the facility. (See additional discussion in Chapter 2, Section 2, Speed.)

Table 3-17 provides minimum design speeds for freeways:

<table>
<thead>
<tr>
<th>Facility</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mainlanes - Urban</td>
<td>50 [80]</td>
</tr>
<tr>
<td>Mainlanes - Rural</td>
<td>70 [110]</td>
</tr>
</tbody>
</table>
Level of Service

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

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

Lane Width and Number

The minimum and usual mainlane width is 12 ft [3.6 m]. The number of lanes required to accommodate the anticipated traffic in the design year is determined by the level of service evaluation as discussed in the *Highway Capacity Manual*. See Table 3-18: Roadway Widths for Controlled Access Facilities and Figure 3-15 for further information.
Shoulders

Continuous surfaced shoulders are provided on each side of the mainlane roadways, both rural and urban, as shown in Figure 3-15. The minimum widths should be 10 ft [3.0 m] on the outside and 4 ft [1.2 m] on the median side of the pavement for four-lane freeways. On freeways of six lanes or more, 10 ft [3.0 m] inside shoulders for emergency parking should be provided. A 10 ft [3.0 m] outside shoulder should be maintained along all speed change lanes with a 6 ft [1.8 m] shoulder considered in those instances where light weaving movements take place. See Table 3-18: Roadway Widths for Controlled Access Facilities and Figure 3-15 for further information.

Medians

The width of the median is the distance between the inside edges of the travel lanes. For depressed freeway sections, medians 76 ft [22.8 m] in width are generally used. Where topography, right-of-
way, or other special considerations dictate, depressed freeway median width may be reduced from 76 ft [22.8 m] to a minimum of 48 ft [14.4 m]. A median width of 24 ft to 30 ft [7.2 m to 9.0 m] is generally used on freeway sections with flush medians. On freeways including six or more travel lanes and a flush 24 ft [7.2 m] median, the resulting section provides for 10 ft [3.0 m] inside shoulders and a usual 2 ft [0.6 m] offset to barrier centerline. See Figure 3-15 for further information.

Because of high speed and volume traffic on urban freeways and the resulting adverse environment for accomplishing construction improvements thereon, it is the usual practice to construct the ultimate freeway section initially. Under those unusual circumstances where future additional lanes will be provided in the median area, the usual median width of 24 ft [7.2 m] should be increased by the appropriate multiple of 12 ft [3.6 m] in anticipation of need for additional lanes. Provisions should be made, or retained, for any future high occupancy vehicle lanes in the median.

At horizontal curves on freeways with narrow medians, a check should be made to insure that the median barrier does not restrict stopping sight distance to less than minimum values.

For information on freeway median crossings, refer to Chapter 7, Emergency Median Openings on Freeways.

**Outer Separation**

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

**Crossing Facilities**

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

- Urban Streets: Table 3-1: Geometric Design Criteria for Urban Streets
- Suburban Roadways: Table 3-5: Geometric Design Criteria for Suburban Roadways
- Rural Two-Lane Highways: Table 3-7. Geometric Design Criteria for Rural Two-Lane Highways and Table 3-8: Width of Travel Lanes and Shoulders on Rural Two-lane Highways
- Multilane Rural Highways: Table 3-12: Design Criteria For Multilane Rural Highways (Non-controlled Access) (All Functional Classes)

The Bridge Project Development Manual should also be referenced for appropriate structure widths.
**Vertical and Clear Zone at Structures**

**Vertical.** All controlled access highway grade separation structures, including railroad underpasses, should provide 16.5 ft [5.0 m] minimum vertical clearance over the usable roadway. Specific railroad company guidelines may also require additional vertical clearance requirements as a function of structure type (Reference applicable railroad company guidelines). Additionally, all Freeways are designated as being on the Texas Highway Freight Network (THFN), thus additional vertical clearance requirements will apply to all freeways for projects Let on September 1, 2020 or later. See Ch. 3, Section 8 for specific requirements.

Structures over the mainlanes of interstate or controlled access highways must meet the minimum vertical clearance requirement except within cities where the 16.5 ft [5.0 m] vertical clearance is provided on an interstate loop around the particular city. Less than 16 ft [4.9 m] vertical clearance on rural interstate and single priority defense interstate routes, including ramps and collector-distributor roads, requires approval through the Design Division with the Federal Highway Administration and/or the Military Traffic Management Command Transportation Engineering Agency (MTMCTEA) of the Department of Defense (DOD).

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

Vertical clearances for pedestrian crossover structures should be approximately 1 ft [0.3 m] greater than that provided for other grade separation structures. This is due to the increased risk of personal injury upon impact by over-height loads and the relative weakness of such structures to resist lateral loads from vehicular impact.

The above-specified clearances apply over the entire width of roadway including usable shoulders and include an allowance of 6 inches [150 mm] for future pavement overlays. It is recognized that it is impractical to arrive at the exact clearance dimensions on the structure plans.

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

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

---

Chapter 3 — New Location and Reconstruction (4R)  
Design Criteria  
Section 6 — Freeways
### Table 3-18: Roadway Widths for Controlled Access Facilities

#### (US Customary)

<table>
<thead>
<tr>
<th>Type of Roadway</th>
<th>Inside Shoulder Width(^2) (ft)</th>
<th>Outside Shoulder Width(^2) (ft)</th>
<th>Traffic Lanes (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mainlanes:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-Lane Divided</td>
<td>4</td>
<td>10</td>
<td>24</td>
</tr>
<tr>
<td>6-Lane or more Divided</td>
<td>10</td>
<td>10</td>
<td>36(^1)</td>
</tr>
<tr>
<td>1-Lane Direct Conn.</td>
<td>2 Rdwy.; 4 Str.</td>
<td>8</td>
<td>14</td>
</tr>
<tr>
<td>2-Lane Direct Conn.</td>
<td>2 Rdwy.; 4 Str.</td>
<td>8</td>
<td>24</td>
</tr>
<tr>
<td><strong>Ramps:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ramps(^2) (uncurbed)</td>
<td>2 Rdwy.; 4 Str.</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>Ramps(^3) (curbed)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

#### (Metric)

<table>
<thead>
<tr>
<th>Type of Roadway</th>
<th>Inside Shoulder Width(^2) (m)</th>
<th>Outside Shoulder Width(^2) (m)</th>
<th>Traffic Lanes (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mainlanes:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-Lane Divided</td>
<td>1.2</td>
<td>3.0</td>
<td>7.2</td>
</tr>
<tr>
<td>6-Lane or more Divided</td>
<td>3.0</td>
<td>3.0</td>
<td>10.81</td>
</tr>
<tr>
<td>1-Lane Direct Conn.</td>
<td>0.6 Rdwy.; 1.2 Str.</td>
<td>2.4</td>
<td>4.2</td>
</tr>
<tr>
<td>2-Lane Direct Conn.</td>
<td>0.6 Rdwy.; 1.2 Str.</td>
<td>2.4</td>
<td>7.2</td>
</tr>
<tr>
<td><strong>Ramps:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ramps(^2) (uncurbed)</td>
<td>0.6 Rdwy.; 1.2 Str.</td>
<td>1.8</td>
<td>2.4</td>
</tr>
<tr>
<td>Ramps(^3) (curbed)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

\(^1\) For more than six lanes, add 12 ft. [3.6 m] width per lane.

\(^2\) If sight distance restrictions are present due to horizontal curvature, the shoulder width on the inside of the curve may be increased to 8 ft. [2.4 m] and the shoulder width on the outside of the curve decreased to 2 ft. [0.6 m] (Rdwy) or 4 ft. [1.2 m] (Str).

\(^3\) The curb for a ramp lane will be mountable and limited to 4 inches [100 mm] or less in height. The width of the curbed ramp lane is measured face to face of curb. Existing curb ramp lane widths of 19 ft. [5.7 m] may be retained.
Frontage Roads

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

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

**Function and Uses**

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

In addition to the above-described purposes of frontage roads, many times they prove advantageous when used as the first stage of construction for an ultimate freeway facility. By constructing frontage roads prior to the mainlanes, interim traffic demands very often can be satisfied and a usable section of highway can be opened to the traveling public at a greatly reduced cost.

**Planning**

Frontage roads may be incorporated into a project at various points during the project development, however, later incorporation of frontage roads will be more difficult. Frontage roads may be included:

- During the planning stage
- Subsequent to the planning stage, and
- After the freeway has been constructed.

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

Changes in control of access must be in accordance with 43 TAC §15.54(d)(4).
As specified in the *ROW Preliminary Procedures for the Authority to Proceed Manual*, subsequent changes in the control of access will be as shown on approved construction plans or as provided in instruments conveying right-of-way on authorized projects, or as may be authorized by Commission Minute Order. Where access is permitted to adjacent properties, ingress and egress will be governed by the issuance of permits to construct access driveway facilities as set forth in established Departmental policy which is designed to provide reasonable access, to insure traffic safety, and preserve the utility of highways.

**Design Speed on Frontage Roads**

Design speeds for frontage roads are a factor in the design of the roadway. For consistency, design speeds should be used that match values used for collector streets or highways. For urban frontage roads, the desirable design speed is 50 mph [80 km/h] and the minimum design speed is 30 mph [50 km/h]. See [Table 3-5: Geometric Design Criteria for Suburban Roadways](#) for design speeds for suburban frontage roads, and [Table 3-6: Minimum Design Speed for Rural Two-lane Highways](#) for rural frontage roads.

**Capacity and Level of Service**

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

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

Research Report 1393-4F contains procedures for the following:

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

**Frontage Road Design Criteria**

Design criteria for urban frontage roads are shown in [Table 3-1: Geometric Design Criteria for Urban Streets](#) using the collector criteria. Design criteria for suburban frontage roads are shown in [Table 3-5: Geometric Design Criteria for Suburban Roadways](#) using the collector criteria. Design
criteria for rural frontage roads are shown in Table 3-19: Design Criteria for Rural Frontage Roads. Clear Zones are given in Table 2-12: Clear Zones.

A frontage road will be designed to provide one-way operation for ultimate build. There may be exceptions in certain isolated instances; however, such exceptions will be considered only where, due to extraordinary circumstances, a one-way pattern would impose severe restrictions on circulation within an area. In those cases where such exceptions are considered, they must be approved by the Design Division at the schematic stage.

Table 3-19: Design Criteria for Rural Frontage Roads

<table>
<thead>
<tr>
<th>Design Speed² (mph)</th>
<th>Min. Width¹ for Future Traffic Volume of</th>
<th>(US Customary)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-400 ADT</td>
<td>400-1,500 ADT</td>
</tr>
<tr>
<td>Lanes (ft.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>25</td>
<td>10</td>
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<td>30</td>
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<td>11</td>
<td>12</td>
</tr>
<tr>
<td>Shoulders (ft)⁴</td>
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<td></td>
</tr>
<tr>
<td>Each Shoulder</td>
<td>2⁵</td>
<td>4</td>
</tr>
<tr>
<td>Two-Way Operation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inside Shoulder</td>
<td>2⁵</td>
<td>2³</td>
</tr>
<tr>
<td>One-Way Operation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outside Shoulder</td>
<td>2⁵</td>
<td>4</td>
</tr>
<tr>
<td>One-Way Operation</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

¹May retain existing paved width on a reconstruction project if total paved width is 24 ft. and operating satisfactorily.

²Use rural collector criteria (Table 3-6) for determining minimum design speed.

³At locations where roadside barriers are provided, use minimum 4 ft. offset from travel lane edge to barrier face.

⁴If the one-way frontage road section contains three or more travel lanes, then minimum inside shoulder width is 8 – 10 ft.
### Table 3-19: Design Criteria for Rural Frontage Roads

(Metric)

<table>
<thead>
<tr>
<th>Design Speed² (km/h)</th>
<th>Min. Width¹ for Future Traffic Volume of</th>
<th>0-400 ADT</th>
<th>400-1,500 ADT</th>
<th>1,500-2,000 ADT</th>
<th>2,000 or more ADT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lanes (m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>3.0</td>
<td>3.0</td>
<td>3.3</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>40</td>
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<td>3.3</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
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<td></td>
</tr>
<tr>
<td>60</td>
<td>3.0</td>
<td>3.3</td>
<td>3.3</td>
<td>3.6</td>
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</tr>
<tr>
<td>70</td>
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<td>3.6</td>
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<tr>
<td>80</td>
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<td>3.3</td>
<td>3.6</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>3.0</td>
<td>3.3</td>
<td>3.6</td>
<td>3.6</td>
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<td>100</td>
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<td>3.6</td>
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<tr>
<td>110</td>
<td>3.3</td>
<td>3.3</td>
<td>3.6</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>3.3</td>
<td>3.6</td>
<td>3.6</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>130</td>
<td>3.3</td>
<td>3.6</td>
<td>3.6</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>Shoulders (m)⁴</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Each Shoulder</td>
<td>0.6³</td>
<td>1.2</td>
<td>2.4</td>
<td>2.4 – 3.0</td>
<td></td>
</tr>
<tr>
<td>Two-Way Operation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inside Shoulder</td>
<td>0.6³</td>
<td>0.6³</td>
<td>1.2</td>
<td>1.2⁴</td>
<td></td>
</tr>
<tr>
<td>One-Way Operation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outside Shoulder</td>
<td>0.6³</td>
<td>1.2</td>
<td>2.4</td>
<td>2.4 – 3.0</td>
<td></td>
</tr>
<tr>
<td>One-Way Operation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

¹May retain existing paved width on a reconstruction project if total paved width is 7.2 m and operating satisfactorily.
²Use rural collector criteria (Table 3-6) for determining minimum design speed.
³At locations where roadside barriers are provided, use minimum 1.2 m offset from travel lane edge to barrier face.
⁴If the one-way frontage road section contains three or more travel lanes, then minimum inside shoulder width is 2.4 – 3.0 m.
Conversion of Frontage Roads from Two-Way to One-Way Operation

Existing frontage roads in some areas are currently operating as two-way facilities. Such two-way operation has the following disadvantages:

- Higher crash rates are normally experienced when the frontage roads are two-way. In large part, this is because of the risk of essentially head-on collisions at the ramp terminals.
- Increased potential for wrong-way entry to the mainlanes.
- The intersections of the frontage roads with the arterials are much more complicated. Left turns from the arterial onto the frontage road must be accommodated from both directions. Accordingly, the signal phasing and sequencing options normally available at signalized diamond interchanges cannot be used.
- The overall traffic-carrying capacity of the frontage roads is substantially less than if the same facility were re-striped for one-way operation.

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

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

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

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

The decision to develop a facility to freeway standards becomes the warrant for providing highway grade separations or interchanges at the most important intersecting roadways (usually arterials and some collectors) and railroads. A grade separation refers to the crossing of two roadways by a physical separation so that neither roadway interferes with the other. An interchange is a grade separation with connecting roadways (ramps, loops, or connections) that move traffic between the intersecting highways.

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

Classifications. Interchanges are classified in a general way, according to the number of approach roadways or intersection legs, as 3-leg, 4-leg and multi-leg interchanges. Through common usage, interchanges are descriptively called “Tee” (or Trumpet) for 3-leg design. Cloverleaf (full or partial) and Diamond for 4-leg, and Directional interchanges with 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 following types of interchanges:

- Three Leg Interchanges, and
- Four Leg Interchanges.

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

Three Leg Interchanges

Three-leg interchanges can take any of several forms, although all of the forms provide connections for the three intersecting highways. Three-leg interchanges should be used only 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 appropriate.

Trumpet. The most widely used 3-leg interchange is the trumpet type, as shown in Figure 3-16. 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 roadway handles higher traffic volume and the loop the lower traffic volume.
Direct. High-type directional 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 are large. They contain more than one structure or, alternatively, a three-level structure. Both variations are illustrated in Figure 3-17.

Four Leg Interchanges

Four-leg interchanges can take a wide variety of forms. The choice of interchange type 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 as follows:

- Diamond Interchanges,
- Cloverleaf Interchanges, and
- Directional Interchanges.
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 be 300 ft [90 m] as a minimum, as shown in Figure 3-18.

![Figure 3-18. Typical Interchange for At-Grade Portion of Diamond Interchange in Urban or Suburban Areas.](image)

The diamond interchange may have several different configurations, as discussed in the following paragraphs and shown in Figure 3-19:

**Conventional diamond without frontage roads.** The conventional diamond (Figure 3-19 A) is the most common application of a diamond interchange. Traffic exits in advance of and near the cross street. Entering vehicles quickly access the freeway beyond or past the cross street. Its disadvantages include exiting vehicles backing up onto the freeway when long queues form on the ramp.
Conventional diamond with frontage roads. The conventional diamond with frontage roads (Figure 3-19 B) is a common variation of a diamond interchange. Traffic exits in advance of and near the cross street. Entering vehicles quickly access the freeway past the cross street. Its disadvantages include 1) exiting vehicles backing up onto the freeway when long queues form on the ramp or frontage road, and 2) most vehicles must go through the intersection to gain access to most frontage road property.

Reverse diamond or X-pattern. The reverse diamond or “X” interchange pattern (Figure 3-19 C) has primary application to locations with significant development along the frontage road. It provides access between interchanges and exiting queues do not back 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.

Spread diamond. The spread diamond (Figure 3-19 D) involves moving the frontage roads outward to provide better intersection sight distance at the cross street and improved operational characteristics with signalized intersections, due to the separation between intersections. However, more additional right-of-way is required, which may limit its usage.

Stacked diamond. Sometimes access to and from the mainlanes is needed on two closely-spaced cross streets. Insufficient distance for consecutive entrance and exit ramps can be resolved by using grade separated ramps, resulting in a “stacked diamond” (Figure 3-19 E).

Split diamond. In some locations, it may be feasible and desirable to “split” the diamond by having one-way streets for the arterial movement (Figure 3-19 F). (This is especially true near central business districts where one-way street systems are common.) However, the split diamond can also be used to accommodate two closely-spaced two-way arterial roadways crossing a freeway.
Three level diamond. In urban areas, where the cross street carries a high volume of traffic, the three-level diamond interchange, illustrated in Figure 3-20, 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 interchange is not usually recommended for use as the ultimate design at the crossing of two controlled access facilities since 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.
Single point diamond. A special type of freeway-to-arterial interchange has received attention during recent years and is worthy of discussion. AASHTO’s *A Policy on Geometric Design of Highways and Streets* refers to it as a “single point diamond” or “single point urban” interchange. In this type of interchange, the freeway mainlanes may go either over or under the crossing arterial and the turn movements occur at-grade on the arterial, as illustrated in Figure 3-21. 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.
Three level stacked diamond. The three-level stacked diamond interchange is also an interchange requiring only one signalized intersection. In a sense, it is a three-level version of the “single point diamond” configuration, as illustrated in Figure 3-22. 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.

As indicated in Figure 3-23, 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.
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 interchanging a freeway 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 (exceed 1200 pcp/h) since loop ramps are limited to one lane of operation and have restricted operating speeds.

Primary disadvantages of the cloverleaf design include the following:

- Large right-of-way requirements,
- Capacity restrictions of loops, especially if truck volumes are significant,
- Short weaving length between loops, and
- Trucks have difficulty with weaves and acceleration.

When used, cloverleaf designs should include collector-distributor roads to provide more satisfactory operations as further noted in the section on Collector-Distributor Roads.
**Full cloverleaf.** The four-quadrant, full cloverleaf, illustrated in Figure 3-24, eliminates all left-turn conflicts through construction of a two-level interchange.

![Full Cloverleaf Interchange](image)

*Figure 3-24. Full Cloverleaf Interchange.*

**Partial cloverleaf.** A cloverleaf without ramps in all four quadrants, illustrated in Figure 3-25, is sometimes used when site controls (such as railroads or streams running parallel to the crossroad) limit the number of loops and/or the traffic pattern is such that the left-turn conflicts caused by the absence of one or more loops are within tolerable limits. With such an arrangement, left-turn conflicts at the ramp intersections require that satisfactory approach sight distance be provided. Several variations on partial cloverleafs are also discussed in the AASHTO’s *A Policy on Geometric Design of Highways and Streets.*
Figure 3-25. Partial Cloverleaf Interchange.

**Directional Interchanges**

Interchanges that use direct or semi-direct connections for one or more left-turn movements are called “directional” interchanges (Figure 3-26). When all turning movements travel on direct or semi-direct ramps or direct connections, the interchange is referred to as “fully directional”. These connections are used for important turning movements instead of loops to reduce travel distance, increase speed and capacity, reduce weaving and avoid loss of direction in traversing a loop. “Fully directional” interchanges are usually justified at the intersection of two freeways.
Figure 3-26. *Four Level Fully Directional Interchange Without Frontage Roads.*

**Four level without frontage roads.** The four-level directional interchange as depicted in Figure 3-26 includes direct connections for all freeway-to-freeway movements, without continuation of any frontage roads through the interchange.

**Five level with frontage roads.** In some instances, it may be desirable to continue the frontage roads through the interchange at the first or second level, producing a five-level directional interchange. Where frontage roads are made continuous through the interchange, the lower three levels are a three-level diamond configuration. Where stage construction is desired, the three-level diamond will adequately serve moderate traffic volumes until the upper two levels of direct connections are constructed to complete the five-level interchange. Figure 3-27 depicts a five level interchange with frontage roads.
Ramps and Direct Connections

This subsection discusses ramps and direct connections and includes information on the following topics:

- General Information,
- Design Speed,
- Horizontal Geometrics,
- Distance Between Successive Ramps,
- Cross Section and Cross Slopes,
- Sight Distance, and
- Metered Ramps.
General Information

All ramps and direct connections should be designed for one-lane operation with provision for emergency parking; however, if the anticipated volume exceeds the capacity of one freeway lane, two-lane operation may be provided with consideration given to merges and additional entry lanes downstream. Several examples of ramps and connecting roadway arrangements are shown in Figures 3-28 through 3-34.

Figure 3-28. (US). Entrance/Exit Ramps For One-Way Frontage Roads.
Figure 3-29. (US). Entrance or Exit Ramps For Two-Way Frontage Roads (Turnaround Provided).
Figure 3-30. (US). Two-Way Frontage Roads Exit and Entrance Ramps (Turnaround Prohibited).
Figure 3-31. (US). Design Details For Ramp Transitions Into Single or Multiple Roadways.
Figure 3-32. (US). Typical Exit Ramps without Frontage Roads.
Figure 3-33. (US). Typical Channelized Exit and Entrance Ramps (Two-Way Frontage Road).
Once ramps have been located on a schematic layout and the same has been exhibited at a public hearing or the design has otherwise become a matter of public record, extreme caution should be exercised in making any subsequent changes in ramp location to better serve areas that may have developed after the original design was determined. In all cases, proposed changes should be submitted to the Design Division, and another public hearing may be required.

Right-side ramps are markedly superior in their operational characteristics and safety to those that leave or enter on the left. With right-side ramps, merging and diverging maneuvers are accomplished into or from the slower moving right travel lane. Since a high majority of ramps are right-side, there is an inherent expectancy by drivers that all ramps will be right-side, and violations of driver expectancy may adversely affect operation and safety characteristics.

Direct access to and from ramps or direct connections can seriously impair safety and traffic operations and, therefore, should not be permitted.
Design Speed

There should be a definite relationship between the design speed on a ramp or direct connection and the design speed on the intersecting highway or frontage road. All ramps and connections should be designed to enable vehicles to leave and enter the traveled way of the freeway at no less than 50 percent (70 percent usual, 85 percent desirable) of the freeway’s design speed. Table 3-20 shows guide values for ramp/connection design speed. 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 design speed for a ramp should not be less than the design speed on the intersecting frontage roads. AASHTO’s *A Policy on Geometric Design of Highways and Streets* provides additional guidance on the application of the ranges of ramp design speed shown in Table 3-20:

Table 3-20: Guide Values for Ramp/Connection Design Speed as Related to Highway Design Speed*

<table>
<thead>
<tr>
<th>Highway Design Speed (mph)</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
<th>55</th>
<th>60</th>
<th>65</th>
<th>70</th>
<th>75</th>
<th>80</th>
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<tbody>
<tr>
<td><strong>Ramp</strong> Design Speed (mph):</td>
<td>-</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Upper Range (85%)</td>
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<td>30</td>
<td>35</td>
<td>40</td>
<td>45</td>
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<td>70</td>
</tr>
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<td>Mid-Range (70%)</td>
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<td>30</td>
<td>33</td>
<td>35</td>
<td>40</td>
<td>45</td>
<td>45</td>
<td>50</td>
<td>55</td>
<td>60</td>
</tr>
<tr>
<td>Lower Range (50%)</td>
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<td>20</td>
<td>23</td>
<td>25</td>
<td>28</td>
<td>30</td>
<td>30</td>
<td>35</td>
<td>40</td>
<td>45</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highway Design Speed (km/h)</td>
<td>50</td>
<td>60</td>
<td>70</td>
<td>80</td>
<td>90</td>
<td>100</td>
<td>110</td>
<td>120</td>
<td>130</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Ramp</strong> Design Speed (km/h):</td>
<td>-</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Upper Range (85%)</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>70</td>
<td>80</td>
<td>90</td>
<td>100</td>
<td>110</td>
<td>120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mid-Range (70%)</td>
<td>30</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>60</td>
<td>70</td>
<td>80</td>
<td>90</td>
<td>100</td>
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<td></td>
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<td>50</td>
<td>50</td>
<td>60</td>
<td>70</td>
<td>80</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* For corresponding minimum radius, see Table 2-6.

**Loops: Upper and middle range values of design speed generally do not apply. The design speed on a loop should be no less than 25 mph [40 km/h] (144 ft. [43 m] minimum radius) based on an $e_{\text{max}}$ of 6%. Particular attention should be given to controlling superelevation on loops due to the tight turning radii and speed limitations.

Horizontal Geometrics

Lane and shoulder widths for ramps and direct connections are shown in Table 3-18.
Figure 3-35 provides design criteria for entrance and exit ramp acceleration, deceleration, and taper lengths; adjustment factors for grade effects are shown in Table 3-14: Speed Change Lane Adjustment Factors as a Function of a Grade.

Exit and entrance ramp typical details are shown in Figure 3-28, Figure 3-29, Figure 3-30, Figure 3-31, Figure 3-32, Figure 3-33, and Figure 3-34.

Grade-separated (braided) entrance and exit ramps should be used only where ramp volumes are considerably greater than frontage road traffic such as where stub frontage roads occur. Where used, the exit ramp desirably should cross the frontage road at approximately 90 degrees to minimize wrong-way entry. Passing should be restricted between the crossroad and the channelized area.

**Figure 3-35.** (US). Lengths of Exit and Entrance Ramp Speed Change Lanes.
Distance between Successive Ramps

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. For analysis of these requirements, see the *Highway Capacity Manual*. Figure 3-36 shows minimum distances between ramps for various ramp configurations.

![Figure 3-36. Arrangements for Successive Ramps.](image)

Cross Section and Cross Slopes

Superelevation rates, as related to curvature and design speed of the ramp or direct connector, are given in Table 3-21. While connecting roadways represent highly variable conditions, as high a superelevation rate as practicable should be used, preferably in the upper half or third of the indicated range, particularly in descending grades. Superelevation rates above 8% are shown in Table 3-21 only to indicate the limits of the range. Superelevation rates above 8% are not recommended and a larger radius is preferable.
The cross slope on portions of connecting roadways or ramps on tangent normally is sloped one way at a practical rate of 1.5% to 2%.

### Table 3-21: Superelevation Range for Curves on Connecting Roadways

#### (US Customary)

<table>
<thead>
<tr>
<th>Radius (ft)</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
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<td>2-10</td>
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<td>-</td>
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<tr>
<td>430</td>
<td>2-4</td>
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<td>540</td>
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<td>5-7</td>
<td>7-9</td>
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<td>2</td>
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</tbody>
</table>

* See Tables 2-6 and 2-7 for design speeds greater than 45 mph.

#### (Metric)

<table>
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<th>Radius (m)</th>
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<td>2</td>
<td>2</td>
<td>2-3</td>
</tr>
</tbody>
</table>

* See Tables 2-6 and 2-7 for design speeds greater than 70 km/h.
The change in pavement edge elevation per given length of connecting roadway or ramp should be that as shown in Table 2-8: Maximum Relative Gradient for Superelevation Transition. The maximum algebraic difference in pavement cross slope at connecting roadways or ramps should not exceed that set forth in Table 3-22:

**Table 3-22: Maximum Algebraic Differences in Pavement Cross Slope at Connecting Roadway Terminals**

<table>
<thead>
<tr>
<th>(US Customary)</th>
<th>(Metric)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed of Exit or Entrance Curve (mph)</td>
<td>Maximum Algebraic Difference in Cross Slope at Crossover Line (%)</td>
</tr>
<tr>
<td>Less than or equal to 20</td>
<td>5.0 to 8.0</td>
</tr>
<tr>
<td>25 to 30</td>
<td>5.0 to 6.0</td>
</tr>
<tr>
<td>Greater than or equal to 35</td>
<td>4.0 to 5.0</td>
</tr>
</tbody>
</table>

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
- Lane balance
- Where two lanes are required by volume, the provision of parallel merging two lanes onto the mainlanes must be provided at the terminal

**Sight Distance**

On all ramps and direct connections, the combination of grade, vertical curves, alignments and clearance of lateral and corner obstructions to vision shall be such as to provide sight distance along such ramps and connections from terminal junctions along the freeway, consistent with the probable speeds of vehicle operation. Sight distance and sight lines are especially important at merge points for ramps and mainlanes or between individual ramps. Table 2-1: Stopping Sight Distance shows recommended stopping sight distances for ramps and direct connections.

The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum stopping sight distance for the freeway design speed, preferably by 25 percent or more. Decision sight distance, as discussed in Decision Sight Distance in Chapter 2, is a desirable goal.
Grades and Profiles

Design controls for crest and sag vertical curves on ramps and direct connectors may be obtained from Figures 2-6 and 2-7. Longer vertical curves with increased stopping sight distances should be provided wherever possible.

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. (Upgrades and Down grades) For certain geometric constraints or topographic conditions, the following grades may be used: Ramp Design Speed 25-30 mph-7% max, 35-40 mph-6% max, and 45 mph or greater-5% max.

Metered Ramps

Where ramps are initially, or subsequently, expected to accommodate metering, the geometric design features shown in Design Criteria for Ramp Metering may be considered. Ramp metering, when properly designed and installed, has been shown to have potential 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 any 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.

Collector-Distributor Roads

A collector-distributor road may be warranted within an interchange, through two adjacent interchanges or continuous for some distance along the freeway through several interchanges. Collector-distributor roads should be provided at all cloverleaf interchanges and particularly at such interchanges on controlled access facilities. A collector-distributor road is designed to meet the following goals:

◆ Transfer weaving from the mainlanes
◆ Provide single-point exits from the main lanes
◆ Provide exit from the main lanes in advance of cross roads.

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 for some distance, should be provided.

Frontage Road Turnarounds and Intersection Approaches

Turnaround lanes are to be provided at all interchanges with major arterials in urban and suburban areas where the freeway lanes are flanked by one-way frontage roads. Turnaround lanes are not to
be provided where two-way frontage roads are used. In urban and suburban areas, overpasses should be arranged so that turnarounds may be added in the future. This includes provisions for end spans and vertical clearance for future turnarounds at overpasses. Underpass situations should also allow for vertical clearances on future elevated turnarounds.

Figure 3-37 shows a typical turnaround at a diamond interchange.

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.

![Figure 3-37. Typical Diamond Interchange with Frontage Road.](image-url)
Section 7 — Freeway Corridor Enhancements

Overview

This section discusses other transportation modes and includes discussions on where to find information on planning and design criteria for these modes.

Freeways with High Occupancy Vehicle Treatments

High Occupancy Vehicles (HOV) lanes are becoming common in urban freeway environment as an approach to reducing congestion and travel times.


Light Rail Transit

Light Rail Transit systems are being considered in some urban environments as an approach to reducing congestion and travel times.

Guidelines for the incorporation of light rail transit systems in the transportation network are given in the Federal Transit Administration publication by Korve, Farran, Mansel, Levinson, Chira-Chaval and Ragland, TCRP Report 17, Integration of Light Rail Transit into City Streets, Transportation Research Board, National Academy Press, Washington, D.C., 1996.
Overview

On September 28, 2017 a memorandum was distributed by Bill Hale, P.E. (Chief Engineer) that provided general guidance for the TxDOT Freight Vertical Clearance Policy. The purpose of the Freight Policy is to make Texas the leader in providing movement of Freight in the Nation. This is a process that should be approached with a well thought out 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 due to low vertical clearances act as Freight bottlenecks; even if there are no immediate plans for new construction or reconstruction of these bridges. Consideration for exchanging overpasses for underpasses should be given as well.

This policy is applicable to projects that meet the following criteria;

- Let on September 1, 2020 or later.
- Designated as being on the THFN as shown on the THFN map maintained by the Transportation Planning and Programming Division (TP&P).
- Bridge new construction or reconstruction; including bridge widening. BMIP and maintenance projects may not be considered bridge reconstruction, but this would need to be handled on a case-by-case basis in consultation with the Bridge Division. Redecking a bridge will also be handled on a case-by-case basis in consultation with the Bridge Division.

Note, the policy does not apply to THFN overpasses, frontage roads, direct connectors off of the THFN, and entrance and exit ramps that include bridge underpasses. The THFN vertical clearance requirements should be considered for bridge and incidental vertical obstruction to the Freight Network that support significant origin/destination locations.

Basic Design Criteria

Vertical Clearance at Structures

Applicable structures on the THFN project must meet the minimum vertical clearance requirement of 18.5 ft.

Minimum vertical clearances for pedestrian crossover structures is 19.5 ft; this is due to the increased risk of personal injury upon impact by over-height loads and the relative weakness of such structures to resist lateral loads from vehicular impact.

The above-specified clearances apply over the entire width of roadway including usable shoulders and include an allowance of 6 inches for future pavement overlays.
Signs, Overhead Sign Bridges (OSB’s), Signals

For the designated THFN, overhead signs shall provide a vertical clearance of not less than 19 feet 6 inches 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 feet above the pavement. See the latest version of the TMUTCD for additional guidance.

Other Overhead Utilities

All overhead utilities over the designated THFN project must meet the requirements specified in the TxDOT ROW Utility Manual, Chapter 3.
Chapter 4 — Non-Freeway Rehabilitation (3R) Design Criteria

Contents:

Section 1 — Purpose
Section 2 — Design Characteristics
Section 3 — Safety Enhancements
Section 4 — Frontage Roads
Section 5 — Bridges, Including Bridge-Classification Culverts
Section 6 — Super 2 Highways
Section 1 — Purpose

Overview

The basic purposes of resurfacing, restoration or rehabilitation (3R) projects are to preserve and extend the service life of existing highways and streets and to enhance safety. Because of limited resources, individual rehabilitation projects may have to be limited in scope in an effort to preserve the mobility function of the entire highway system. The scope of 3R projects varies from thin overlays and minor safety upgrading to more complete rehabilitation.

3R projects are those which address pavement needs and/or deficiencies and which substantially follow the existing horizontal and vertical alignment. They differ from reconstruction projects in that reconstruction projects substantially deviate from the existing horizontal and/or vertical alignment and/or add capacity.

Design Guidelines

Design guidelines for 3R projects have been developed to allow greater design flexibility. At the District’s option, design values above those presented in this chapter may be used.

These guidelines offer sufficient flexibility to ensure cost effective design and further compliance with the program goals of preserving and extending the service life and enhancing safety. While safety may not be the primary reason for initiating a 3R project, highway safety is an essential element of all projects. These 3R projects are to be developed in a manner which identifies and incorporates appropriate safety enhancements.

For the purpose of 3R projects, current average daily traffic (ADT) volumes of less than 1500 are defined as low traffic volume roadways.
Section 2 — Design Characteristics

Pavement Design

Pavement rehabilitation includes all pavement-related work undertaken to extend the service life of an existing facility. This includes placement of additional surfacing material and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural and/or functional adequacy. The following are some examples of pavement rehabilitation work:

- Resurfacing to provide improved structural capacity and/or serviceability,
- Removing and replacing deteriorated materials,
- Replacing or restoring malfunctioning joints,
- Reworking or strengthening of bases and subbases,
- Recycling existing materials, and
- Adding underdrains.

The existing pavement condition and deficiencies should be identified for 3R projects. Design strategies selected to correct deficiencies will vary from seal coats to overlays to complete pavement structure reconstruction. Projects that consist only of seal coats or overlays, and do not evaluate the project according to the additional guidelines presented in this chapter, are not eligible for rehabilitation funding.

Reference can be made to the Pavement Design Guide for additional information related to pavement rehabilitation.

Geometric Design

Geometric design guidelines are provided for the following roadways in the tables indicated.

- Rural multilane highways, Table 4-1,
- Rural two-lane highways, Table 4-2,
- Urban streets, Table 4-3,
- Rural frontage roads, Table 4-4, and
- Urban frontage roads, Table 4-5.

For the purposes of measuring Bridge Width on bridge structures, the bridge width on bridge structures without curbs is measured to the nominal face of rail. Reference the TxDOT Bridge Railing Manual and Bridge Railing Standards for the nominal widths of specific...
rail types and additional guidance. For Bridges with curbs, the bridge width is measured to the face of curb.

Table 4-1: 3R Minimum Design Guidelines for Rural Multilane Highways (Nonfreeway)a

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Highway Class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6-Lane Divided</td>
</tr>
<tr>
<td>Design Speedb</td>
<td>50 mph</td>
</tr>
<tr>
<td>Lane Width</td>
<td>11 ft</td>
</tr>
<tr>
<td>Outside Shoulder</td>
<td>4 ft</td>
</tr>
<tr>
<td>Inside Shoulder</td>
<td>4 ft</td>
</tr>
<tr>
<td>Turn Lane Widthc</td>
<td>10 ft</td>
</tr>
<tr>
<td>Clear Zonee</td>
<td>16 ft</td>
</tr>
<tr>
<td>Bridgesd; Width to be retained</td>
<td>42 ft</td>
</tr>
</tbody>
</table>

a These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area or to provide operational improvements at specific locations.

b Considerations in selecting design speeds for the project should include the roadway alignment characteristics as discussed in this chapter.

c For two-way left turn lanes, 11 ft – 14 ft usual.

d Where structures are to be modified, bridges should meet approach roadway width as a minimum. (Approach roadway width is the total width of the lanes and shoulders.) Greater bridge widths may be appropriate if the rehabilitation project increases roadway life significantly or if higher design values are selected for the remainder of the project. Existing structure widths less than those shown may be retained if the total lane width is not reduced across or in the vicinity of the structure.

e For low-speed rural collectors, and all rural local roads, a clear zone of 10 ft is allowable in constrained circumstances. The clear zone is measured from the edge of travel lane to the obstruction.

Table 4-2: 3R Minimum Design Guidelines for Rural Two-Lane Highwaysa

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Current Average Daily Traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 – 400</td>
</tr>
<tr>
<td>Design Speedb</td>
<td>30 mph</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>0 ft</td>
</tr>
</tbody>
</table>
### Table 4-2: 3R Minimum Design Guidelines for Rural Two-Lane Highways<sup>a</sup>

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Guideline</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane Width</td>
<td>10 ft</td>
</tr>
<tr>
<td>Surfaced Roadway</td>
<td>20 ft</td>
</tr>
<tr>
<td>Turn Lane Width&lt;sup&gt;c&lt;/sup&gt;</td>
<td>10 ft</td>
</tr>
<tr>
<td>Clear Zone&lt;sup&gt;f&lt;/sup&gt;</td>
<td>7 ft</td>
</tr>
<tr>
<td>Bridges&lt;sup&gt;d&lt;/sup&gt;: Width to be retained</td>
<td>20 ft</td>
</tr>
</tbody>
</table>

<sup>a</sup> These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area or to provide operational improvements at specific locations.

<sup>b</sup> Considerations in selecting design speeds for the project should include the roadway alignment characteristics as discussed in this chapter.

<sup>c</sup> For two-way left turn lanes, 11 ft – 14 ft usual.

<sup>d</sup> Where structures are to be modified, bridges should meet approach roadway width as a minimum. (Approach roadway width is the total width of the lanes and shoulders.) Greater bridge widths may be appropriate if the rehabilitation project increases roadway life significantly or if higher design values are selected for the remainder of the project. Existing structure widths less than those shown may be retained if the total lane width is not reduced across or in the vicinity of the structure.

<sup>e</sup> For current ADT exceeding 2000, minimum width of bridge to be retained is 28 ft.

<sup>f</sup> The clear zone is measured from the edge of travel lane to the obstruction.

---

### Table 4-3: 3R Minimum Design Guidelines for Urban Streets<sup>a</sup> All Functional Classes

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Guideline</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed&lt;sup&gt;b&lt;/sup&gt;</td>
<td>30 mph</td>
</tr>
<tr>
<td>Lane Width</td>
<td>10 ft</td>
</tr>
<tr>
<td>Turn Lane Width&lt;sup&gt;c&lt;/sup&gt;</td>
<td>10 ft</td>
</tr>
<tr>
<td>Parallel Parking Lane Width</td>
<td>7 ft</td>
</tr>
<tr>
<td>Curb Offset</td>
<td>0 ft</td>
</tr>
<tr>
<td>Shoulders&lt;sup&gt;d,e&lt;/sup&gt;</td>
<td>2 ft</td>
</tr>
<tr>
<td>Clear Zone</td>
<td>To back of curb or outside edge of shoulder</td>
</tr>
<tr>
<td>Bridges: Width to be retained</td>
<td>Approach roadway, not including shoulders</td>
</tr>
</tbody>
</table>
Where the existing highway features comply with the design values given in this chapter, the designer may choose not to modify these features. However, where the existing features do not meet these values, upgrading should be to the values shown in this chapter. These values are intended for use on typical rehabilitation projects. The designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area or to provide operational improvements at specific locations.

**Alignment**

Typically, 3R projects will involve minor or no change in either vertical or horizontal alignment. However, flattening of curves or other improvements may be considered 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, improvements in superelevation may also be a consideration.

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 upgrades are needed, but those upgrades are not included in the 3R project. See Chapter 1, Section 2 for specific geometric criteria requiring either a design exception or design waiver.

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**Table 4-3: 3R Minimum Design Guidelines for Urban Streets\textsuperscript{a} All Functional Classes**

<table>
<thead>
<tr>
<th>(US Customary)</th>
</tr>
</thead>
<tbody>
<tr>
<td>\textsuperscript{a} These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area or to provide operational improvements at specific locations.</td>
</tr>
<tr>
<td>\textsuperscript{b} Considerations in selecting design speeds for the project should include the roadway alignment characteristics as discussed in this chapter.</td>
</tr>
<tr>
<td>\textsuperscript{c} For two-way left turn lanes, 11 ft – 14 ft usual.</td>
</tr>
<tr>
<td>\textsuperscript{d} Minimally 1 ft of shoulder surfaced where lane width is 10 ft thereby providing a 22 ft surfacing width.</td>
</tr>
<tr>
<td>\textsuperscript{e} Applicable to uncurbed streets.</td>
</tr>
</tbody>
</table>
Design Speed

The reconstruction of horizontal and vertical alignments should be considered when the suggested design speed of the particular roadway in question is not consistent with the existing geometrics. For rehabilitation purposes, the suggested minimum design speed for rural multilane highways is 50 mph [80 km/h]. The suggested minimum design speed for high volume rural two-lane highways and high volume rural frontage roads is 40 mph [60 km/h]. The suggested minimum design speed for low volume rural two-lane highways, low volume rural frontage roads, urban streets, and urban frontage roads is 30 mph [50 km/h].

This does not imply that roadways with alignments falling below these current design speed values are unsafe. Rather, these roadways were usually designed to values considered current at the time of construction or at a time when alignment criteria was nonexistent for that particular type of roadway. These roadways may experience enhanced safety and improved traffic operations if the proposed rehabilitation project can cost effectively make alignment improvements.

For roadways not meeting the suggested 3R design speeds, an evaluation should be done to examine high frequency crash locations to determine whether cost effective alignment revisions can be accomplished with the resources available.

Side and Backslopes

Existing side and backslopes usually should be retained except where crown widening or grade changes create conditions that dictate otherwise.

Lane Widths

Consideration should be given to increasing lane widths to 12 ft [3.6 m] in conjunction with rehabilitation projects where the highway is a high volume route utilized extensively by large trucks. This consideration should be factored in along with all of the other normal considerations that determine the scope of a project, including expected service life of the proposed rehabilitation work, long range plans for the route and the design standards of other nearby segments on the route.
Section 3 — Safety Enhancements

Overview

Resurfacing, restoration, and rehabilitation projects are to be developed in a manner which identifies and incorporates appropriate safety enhancements. Engineering judgement will have to play a part in determining the extent to which safety improvements can reasonably be made with the limited resources available. Traffic volumes are an important factor to be considered when evaluating cost-effectiveness of potential safety improvements. Typically, safety improvements are the most cost effective on roadways with higher traffic volumes. This should not imply that safety enhancements on lower traffic volume roadways are not to be considered. Even relatively low-cost incremental safety enhancements can significantly reduce crash frequency and/or severity.

Safety Design

Transportation Research Board Special Report 214, Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation, describes a safety conscious design process for 3R projects as follows:

“Significant improvements in safety are not automatic by-products of RRR projects; safety must be systematically engineered into each project. To do this, highway designers must deliberately seek safety opportunities specific to each project and apply sound safety and traffic engineering principles. Highway agencies must strengthen safety considerations at each major step in the design process, treating safety as an integral part of design and not as a secondary objective. These actions require that highway agencies devote greater resources to RRR project design. . .”

Special Report 214 offers suggestions for considering project specifics very early in the 3R design process. These suggestions are paraphrased as follows:

- At the beginning of 3R project design, highway designers should assess existing physical and operational conditions related to safety.
  - Gather data to identify specific safety problems that might be corrected and compare this data with the system-wide performance of similar highways.
  - Conduct a site inspection using experienced personnel to recognize the opportunities for safety improvements within the common operating conditions of that individual roadway.
  - Determine and verify existing geometry such as roadway widths, horizontal and vertical curvature, intersection layout, and other geometrics specific to the roadway section being examined.
- In addition to pavement repairs and geometric improvements, designers of 3R projects should consider incorporating other intersection, roadside, and traffic control improvements that may enhance safety, including wildlife crossing structures.
• At horizontal curves where reconstruction cannot be accomplished, designers should evaluate less costly safety measures such as widening narrow pavements, flattening steep side slopes, removing or relocating roadside obstacles, and meeting TMUTCD requirements of installing horizontal alignment warning traffic control devices, delineators and pavement markings. Additionally, High Friction Surface Treatment (HFST) can be considered to address deficiencies in curve radius or superelevation.

• Whether or not bridge widening is necessary on a particular project, designers should routinely evaluate guardrail installations at the bridge approaches, existing bridge rails for rehabilitation or replacement, and approach signing or delineation for inclusion in the project if appropriate and cost effective.

◆ Before developing construction plans and specifications, designers should document the project evaluation and give the design criteria which will be used to produce the final rehabilitation project.

Other methods have been successfully used to identify potential safety problems. These may be used at the designer’s option to meet the particular needs of the project.

◆ Maintenance personnel are familiar with a particular route and can point out problem areas to the designer based on their experiences. These individuals frequently “work” crash locations and are called upon to perform corrective work necessitated by crashes. Carcass data collection information can be used determine if there are natural pathways for wild-life movement.

◆ A crash analysis can be conducted. Refer to Highway Safety Manual (HSM) Chapter 5 for diagnosis procedures to identify causes of collision, safety concerns and crash patterns. Coordinate with Design Division—Traffic Simulation and Safety Analysis Section for further guidance. Chapter 2, Section 3 of the Traffic Safety Program Manual also includes procedures and guidance on producing a crash analysis report. Additional information is available from the Traffic Safety Division. Coordinate with District Traffic personnel for more information and coordinating traffic safety and operational improvements. District Traffic personnel have the expertise to suggest corrective safety countermeasures that should be designed into the 3R project.

◆ A hot spot analysis of crash data to determine factors for safety problems, including animal-vehicle crashes.

◆ Consult the District Safety Plan
A summary of the safety and operations evaluation should be included in the project files and be made available during plan review. This evaluation should document the presence, or absence, of any major deficiencies which may contribute to operational issues and frequency and/or severity. This evaluation should be initially considered when scoping work in order that corrective measures may be taken where practicable.

**Project Specific Design Information**

The **Project Specification Design Information** has been developed to assist in the project evaluation and provide one possible outline for file documentation.

For individual project evaluation:

- 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 are the design guidelines given in this chapter which are applicable to this project?
- What are the design values present 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 are the ADT and the character (truck %, recreational use, local traffic, etc.) of the traffic using the roadway?
- What is the crash history (type, severity, conditions, etc.) of the entire project and at any specific locations which require the individual evaluation of design elements?
- What is the compatibility of the proposed design with adjacent sections of the roadway?

For specific design elements which require individual evaluation prior to final project design:

- What length and percentage of the project is affected by the design elements in question?
- What is the comparative cost of the given design guideline versus the proposed design element in terms of construction, right-of-way availability, project delay, environmental impacts, etc.?
- What is the long term effect of using the design element selected in terms of capacity and level of service?
- If other design elements required individual evaluation, what is believed to be the cumulative effect of these design elements on the safety and operation of the proposed facility?

**Basic Safety Improvements**

Basic safety improvements will be required for all 3R projects. Basic safety improvements are defined as upgrading guardrail to current standards, providing signing and pavement markings in accordance with the **Texas Manual on Uniform Traffic Control Devices** and the Traffic
Safety Division’s Traffic Engineering Standard Sheets, providing a skid resistant surface, and safety treating cross drainage pipe culverts 36 inches [0.9 m] in diameter or smaller that are inside the clear zones given in this chapter. Other safety improvements to consider include treatment of nonstandard mailbox supports, nonstandard luminaire supports, and nonstandard sign supports that are inside the suggested clear zones. Consideration may also be necessary for trees, utility poles or other obstacles where these features are indicated significantly in a crash evaluation.

**Guardrails.** Guardrails shall be upgraded to current hardware standards. Connections to structures, post spacing and end treatments shall meet current design practices. Where guardrail height is 3 inches [75 mm] or more too high or too low, corrections in height are required. Guardrail lengths will generally be designed to requirements given in [Determining Length of Need of Barrier](#) in Appendix A.

All guardrail that is not needed should be removed. Guardrail also should be removed where obstacles being shielded may be cost effectively design treated (removed, made yielding, etc.).

**Headwalls.** Headwalls on small (36 inches [0.9 m] or less) cross drainage pipe culverts that are inside the clear zones given in this chapter should be removed and sloping (1V:3H or flatter) culvert ends that blend with existing side slopes should be installed. Where located behind guardrail, these culvert ends should be safety treated and guardrail removed where there are no other obstructions involved. Where guardrail is required for shielding other obstacles, headwalls behind guardrail need not be safety treated. Also, where other non-removable, non-treatable obstacles are present near these culvert ends, culverts need not be treated.

**Other Safety Enhancements**

**Cross drainage box and pipe culverts.** Cross drainage box and pipe culverts greater than 36 inches [0.9 m] may remain as they exist where the clear zones given in this chapter are satisfied. Where the clear zones given in this chapter are not met, safety treatment (grates, extension, or guardrail) will be required. Where the culvert end creates a safety obstacle that is out of context with the remaining portion of the project, even though it meets clearances, consideration should be given to safety treatment. On the other hand, where other non-removable and non-treatable obstacles are located near culvert ends, treatment of culvert ends would be out of context with the immediate area, and guardrail or non-treatment may be the only choices.

**Culverts.** For culvert spans from 3 ft [0.9 m] to 5 ft [1.5 m] and heights up to 5 ft [1.5 m] that need to be safety treated, the 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 inlet restrictions (entrance loss coefficients) should be evaluated as to their effects on hydraulics. If necessary, reference can be made to the Hydraulic Design Manual for entrance loss coefficients with various configurations as well as other hydraulic design information.
**Driveway embankments and pipes.** Treatment of driveway embankments and pipes will be required 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, more significant geometric and safety related improvements may be appropriate.
Section 4 — Frontage Roads

Overview

Table 4-4: 3R Design Guidelines for Rural Frontage Roads and Table 4-5: 3R Design Guidelines for Urban Frontage Roads show geometric design guidelines for 3R projects on rural and urban freeway frontage roads. These guidelines are acceptable for those projects involving either rehabilitation of only the frontage road or rehabilitation of the frontage road in conjunction with rehabilitation of the freeway mainlanes. It is not the intent of these 3R frontage road design guidelines to be used when a freeway section is reconstructed from right of way line to right of way line even though no additional frontage road lanes are added. Complete frontage road reconstruction projects should reference the applicable reconstruction guidelines for the appropriate criteria.

Frontage roads are built in some locations initially in a phased construction sequence with the mainlanes to be built when traffic conditions warrant. If the frontage road is serving as the principal roadway pending future mainlane construction, the 3R design guidelines for rural multilane highways would be applicable for rehabilitation work on these facilities.

Table 4-4: 3R Design Guidelines for Rural Frontage Roads

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Current Average Daily Traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 - 1500</td>
</tr>
<tr>
<td>Design Speed</td>
<td>30 mph</td>
</tr>
<tr>
<td>Lane Width</td>
<td>10 ft</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>1 ft</td>
</tr>
<tr>
<td>Two-Way Operation</td>
<td></td>
</tr>
<tr>
<td>Inside Shoulder Width</td>
<td>1 ft</td>
</tr>
<tr>
<td>One-Way Operation</td>
<td></td>
</tr>
<tr>
<td>Outside Shoulder Width</td>
<td>1 ft</td>
</tr>
<tr>
<td>One-Way Operation</td>
<td></td>
</tr>
<tr>
<td>Clear Zone</td>
<td>7 ft</td>
</tr>
<tr>
<td>Bridges(^h): Width to be retained</td>
<td>24 ft</td>
</tr>
</tbody>
</table>
These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area, or to provide operational improvements at specific locations.

Where structures are to be modified, bridges should meet approach roadway width as a minimum. (Approach roadway width is the total width of the lanes and shoulders.) Greater bridge widths may be appropriate if the rehabilitation project increases roadway life significantly or if higher design values are selected for the remainder of the project. Existing structure widths less than those shown may be retained if the total lane width is not reduced across or in the vicinity of the structure.

For current ADT exceeding 2000, minimum width of bridge to be retained is 28 ft.

**Table 4-4: 3R Design Guidelines for Rural Frontage Roads**

<table>
<thead>
<tr>
<th>Design Element</th>
<th>All Traffic Volumes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed</td>
<td>30 mph</td>
</tr>
<tr>
<td>Lane Width</td>
<td>10 ft</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>0 ft inside</td>
</tr>
<tr>
<td></td>
<td>2 ft outside</td>
</tr>
<tr>
<td>Clear Zone</td>
<td>Back of curb or edge of shoulder</td>
</tr>
<tr>
<td>Curb Offset</td>
<td>1 ft either side</td>
</tr>
<tr>
<td>Bridges: Width to be retained</td>
<td>Approach roadway not including shoulders</td>
</tr>
</tbody>
</table>

**Table 4-5: 3R Design Guidelines for Urban Frontage Roads**

<table>
<thead>
<tr>
<th>Design Element</th>
<th>(US Customary)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed</td>
<td>30 mph</td>
</tr>
<tr>
<td>Lane Width</td>
<td>10 ft</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>0 ft inside</td>
</tr>
<tr>
<td></td>
<td>2 ft outside</td>
</tr>
<tr>
<td>Clear Zone</td>
<td>Back of curb or edge of shoulder</td>
</tr>
<tr>
<td>Curb Offset</td>
<td>1 ft either side</td>
</tr>
<tr>
<td>Bridges: Width to be retained</td>
<td>Approach roadway not including shoulders</td>
</tr>
</tbody>
</table>

These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area, or to provide operational improvements at specific locations.
Section 5 — Bridges, Including Bridge-Classification Culverts

Overview

Where minimum bridge widths exist, it is generally expected that no additional structural work will be necessary. However, existing conditions such as deficient railing (pre-1964 rails are typically in this category), deteriorated deck, or a structure which has an unsafe load carrying capability may require additional structural work. In such cases, the Bridge Division should be consulted 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, roadways with low traffic volumes should have a crash evaluation conducted on structures with railings which do not match current standard railing details. It is important that bridges with these railings be evaluated on an individual basis. If the evaluation indicates continuing satisfactory performance and the railing is in good repair, these railings may be retained on low volume roadways.

Additional information on bridge rehabilitation may be obtained by referencing the Bridge Project Development Manual.

Note that additional vertical clearance requirements may apply to bridge projects on the Texas Highway Freight Network (THFN) as specified in Chapter 3, Section 8.
Section 6 — Super 2 Highways

Overview

A Super 2 highway is where a periodic passing lane is added to a two-lane rural highway to allow passing of slower vehicles and the dispersal of traffic platoons. The passing lane will alternate from one direction of travel to the other within a section of roadway allowing passing opportunities in both directions. A Super 2 project can be introduced on an existing two-lane roadway where there is a significant amount of slow moving traffic, limited sight distance for passing, and/or the existing traffic volume has exceeded the two-lane highway capacity, creating the need for vehicles to pass on a more frequent basis.

Widening of the existing pavement can be symmetric about the centerline or on one side of the roadway depending on right of way (ROW) availability and ease of construction.

Some issues to consider when designing a Super 2 project:

- Existing ROW width considerations must be analyzed to determine feasibility of upgrading to a Super 2.
- Consider providing a left turn lane if a significant traffic generator falls within the limits of a Super 2.
- Consider providing full shoulders (8'-10') in areas with high driveway density.
- The location of large drainage structures and bridges should be evaluated 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 closing 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.
- When evaluating the termination of a passing lane at an intersection, consideration should be given to traffic operations turning and weaving movements, and intersection geometrics. If closure of the passing lane at the intersection would result in significant operational lane weaving, then consideration should be given to extending the passing lane beyond the intersection.
- Allow adequate distance (recommend stopping site distance) between the end of a lane closure taper and a constraint such as metal beam guard fence, a narrow structure, or major traffic generator.
- Consider providing the passing lane in the direction leaving an incorporated area for potential platoons generated in the urban area.
**Basic Design Criteria**

Recommended design values are shown in Table 4-6.

**Table 4-6: Design Criteria**

<table>
<thead>
<tr>
<th></th>
<th>Minimum</th>
<th>Desirable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed</td>
<td>See Table 4-2</td>
<td></td>
</tr>
<tr>
<td>Clear Zone</td>
<td>See Table 4-2</td>
<td></td>
</tr>
<tr>
<td>Lane Width</td>
<td>11 ft</td>
<td>12 ft</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>3 ft(^a)</td>
<td>8 - 10 ft</td>
</tr>
<tr>
<td>Passing Lane Length</td>
<td>1 mi</td>
<td>1.5 - 2 mi(^b)</td>
</tr>
</tbody>
</table>

\(^a\): Where ROW is limited
\(^b\): Longer passing lanes are acceptable, but not recommended more than 4 miles. Consider switching the direction if more than 4 miles.

The length for opening a passing lane (Figure 4-1) should be based on the following:

\[
L = \frac{WS}{2},
\]

Where

- \(L\) = Length of taper,
- \(W\) = Lane width, and
- \(S\) = Posted speed.

The taper length for closing a passing lane (Figure 4-1) should be based on:

\[
L = WS,
\]

Where

- \(L\) = Length of taper,
- \(W\) = Lane width, and
- \(S\) = Posted speed.
Figure 4-1. Opening and Closing a Passing Lane

When switching the passing lane from one direction to another (closing the passing lane in each direction), provide a taper length from each direction based on \( L = WS \), with a minimum 50 ft buffer (stopping sight distance (SSD) desirable) between them. (Figure 4-2).

Figure 4-2. Closing the Passing Lane from One Direction to Another

When opening a passing lane in each direction (Figure 4-3), provide a taper length based on \( L = WS/2 \).

Figure 4-3. Opening the Passing Lane from One Direction to Another

When widening to the outside of the roadway to provide a passing lane opportunity (Figure 4-4), provide an opening taper length based on \( L = WS/2 \) and a closing taper length based on \( L = WS \).
Figure 4-4. Separated Passing Lanes with Widening to the Outside of Roadway

Passing lanes in each direction may overlap if ROW is sufficient (Figure 4-5).

Provide an opening taper length based on $L = WS/2$ and a closing taper length based on $L=WS$.

Figure 4-5. Side by Side Passing Lanes
Chapter 5 — Non-Freeway Resurfacing or Restoration Projects (2R)

Contents:

Section 1 — Overview
Section 1 — Overview

Guidelines

The following guidelines apply to non-freeway resurfacing or restoration (2R) projects which are not on National Highway System (NHS) routes, and have current average daily traffic (ADT) volumes of 2500 per lane and less. Projects with current average daily traffic (ADT) volumes greater than 2500 per lane and projects which are on NHS system routes may not be designed to 2R guidelines.

These guidelines should also be used in determining design scope and estimating cost for individual candidate projects whenever a restoration program is being developed. Preliminary structural planning should be coordinated with the Bridge Division.

Definition. Restoration projects are defined as work performed to restore pavement structure, riding quality, or other necessary components, to their existing cross section configuration. The principal purposes of these projects are surfacing and repair of the pavement structure. The addition of through travel lanes is not permitted under a restoration project. The addition of continuous two-way left-turn lanes, acceleration/deceleration lanes, turning lanes, and shoulders are acceptable as restoration work as long as the existing through lane and shoulder widths are maintained as a minimum. The restoration work may include upgrading roadway components as needed to maintain the roadway in an acceptable condition.

Upgrading. Where the work is cost effective and funds are sufficient to upgrade to reconstruction or rehabilitation design criteria without jeopardizing district priorities for other restoration work, development of projects to higher criteria may be done at the district’s discretion.

Crash Analysis. A crash analysis should be conducted for 2R projects. Any specific areas involving high crash frequencies will be reviewed and corrective measures taken where appropriate. In addition to a formal analysis of crash data, Chapter 4, Section 3 lists several methods that have been used successfully to identify potential crash problems.
Chapter 6 — Special Facilities

Contents:

Section 1 — Off-System Bridge Replacement and Rehabilitation Projects
Section 2 — Historically Significant Bridge Projects
Section 3 — Texas Parks and Wildlife Department (Park Road (PR) and Park and Wildlife Road (PW)) Projects
Section 4 — Bicycle Facilities
Section 1 — Off-System Bridge Replacement and Rehabilitation Projects

Project Conditions

This section provides design guidance for projects meeting all of 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 class of highway should be utilized. For off-system bridge projects, current ADT may be used with the appropriate class of highway (i.e., enter tables, charts, or figures with current ADT substituted for future ADT). Where significant traffic growth is expected or the roadway will be widened in the near future, the use of future ADT for design purposes is encouraged.

For more information on the off-system bridge replacement and rehabilitation program, refer to the Bridge Project Development Manual.

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

Design Values

Design values selected for a particular project should satisfy and preferably exceed the values shown below. Selected design values should be consistent and compatible with the prevalent design features on the existing off-system roadway. If the route has the potential for significant ADT increases in the near future, or if the character of the traffic is not local, design requirements for the appropriate class of highway should be used.

- Minimum Design Speed: Meet or improve conditions that are typical on the remainder of the roadway.
- Vertical Curvature, Minimum K values: Meet or improve conditions that are typical on the remainder of the roadway.
- Horizontal Curvature: Meet or improve conditions that are typical on the remainder of the roadway.
- Minimum Superelevation: Meet or improve conditions that are typical on the remainder of the roadway.
- Maximum Grades: Meet or improve conditions that are typical on the remainder of the roadway.
◆ Minimum Structure Width, Face to Face of Rail: 24 ft [7.2 m].

◆ Bridge End Guard Fence:
  • Minimum Conditions – Transition section plus end treatment.
  • If an intervening roadway or driveway prevents usual placement of guard fence, a guard fence radius may be used provided the approach represents a low speed condition.

◆ Approach Roadway:
  • For minimum length of 50 ft [15 m] adjacent to the bridge end, the roadway crown should match clear width across structure (24 ft [7.2 m]) plus additional width to accommodate approach guard fence.
  • An appropriate transition (minimum length 50 ft [15 m]) to county road width should be made in the sections of approach roadway located at the federal project extremities.
  • If roadway surfacing is included, a minimum of 20 ft [6 m] surfacing width should be used for the 50 ft [15 m] roadway section adjacent to the bridge.

◆ Traffic Control:
  • When provided, and to the extent practical, detours should match existing county road design features. Design details for detours should be shown in the plans and on the preliminary layouts.
  • Traffic control devices should be in conformance with the Texas Manual on Uniform Traffic Control Devices, and details should be included in the plans.
Section 2 — Historically Significant Bridge Projects

Reference for Procedures

Historically significant bridges command importance and a place in the engineering and cultural heritage of this nation. Federal law requires these bridges be given special consideration, where practical and feasible, toward their preservation in the course of bridge replacement or bridge rehabilitation/improvement projects.

Reference can be made to the *Historic Bridge Manual* for procedures that should be used when developing projects that involve historic bridges.
Section 3 — Texas Parks and Wildlife Department (Park Road (PR) and Park and Wildlife Road (PW)) Projects

Working Agreements

According to Acts 1995, 74th Leg., Ch. 445, §1, the Texas Department of Transportation shall construct, repair and maintain roads in and adjacent to state parks, state fish hatcheries, state wildlife management areas, and support facilities for parks, fish hatcheries, and wildlife management areas.

In response to this legislation, a memorandum of agreement between TxDOT and the Texas Parks and Wildlife Department (TPWD) was established. This memorandum of agreement states that TPWD is to provide TxDOT with current design standards for TPWD facilities.

Accordingly, TPWD facilities, which are those designated as Park and Wildlife Roads (PW), are to be designed based on the criteria and guidance given in the current publication of the Texas Parks and Wildlife Department Design Standards for Roads and Parking.

Park Roads (PR) that lead to or enter a state park, and are designated on the State Highway System, are to be designed based on the criteria and guidance given in the current TxDOT Roadway Design Manual. For roads designated as PR where constrained site conditions, parking, or high pedestrian usage warrant a lower speed; a design speed of no lower than 20 mph may be used.
Section 4 — Bicycle Facilities

Overview

The Texas Legislature has directed TxDOT, in Texas Transportation Code §201.902, to enhance the use of the state highway system by bicyclists. Administrative rules adopted by the commission in 43 TAC §25.50–25.55 affirm TxDOT’s commitment to integrating this mode of travel into project development.

Guidance for Bicycle Facilities

The *AASHTO Guide for the Development of Bicycle Facilities* is the guide for planning, design, construction, maintenance, and operation of bicycle facilities. There are three types of bicycle facilities described in the guide. These are bicycle lanes, shared lanes, and shared use paths. A bicycle lane is defined as a portion of a roadway which has been designated by striping, signing and/or pavement markings for the preferential or exclusive use of bicyclists. For a striped bicycle lane, the clear width is 4 ft [1.2m] minimum and 5' [1.5m] desirable (measured from the lane stripe to the gutter joint or 1 ft [.3m] from the nominal face of curb on a monolithic curb). A shared lane is a 14 ft [4.2m] lane (measured from the lane stripe to the gutter joint or 1 ft [.3m] from the nominal face of curb on a monolithic curb), that is shared by motorists and bicyclists that should have bicycle signing and pavement markings. A shared use path is defined as a bikeway that is physically separated from motorized vehicular traffic by an open space or barrier, either within the highway right of way or within an independent right of way, that can also be used by pedestrians, skaters, joggers, wheelchairs, and other non-motorized users. A shared use path is generally preferred over a bicycle lane or shared lane because the physical separation reduces possible conflicts with vehicles. If a shared use path is provided, an additional bicycle lane or shared lane is not needed. If a shared use path is not feasible, a bicycle lane is generally preferred over a shared lane.

Due to concerns of minimal passing distance and tendency for motor vehicles to increase speeds when wide outside lanes are used, wide curb lanes are not recommended as a strategy to accommodate bicycling except as an interim treatment for retrofits where an existing road is being re-striped and all other travel lanes have been narrowed to the minimum widths. On high speed roadways, shared lanes are not appropriate and should be avoided.
Design Exceptions and Design Waivers for Bicycle Facilities

Design exceptions will be required for bicycle lanes, or shared lanes that do not meet the minimum requirements in the AASHTO Guide for the Development of Bicycle Facilities. For new shared lanes, the minimum lane width shall be 14 ft [4.2 m]. For new bicycle lanes, the clear width is 4 ft ([1.2m] minimum and 5' [1.5m] desirable. Proposed widths less than these will require a design exception.

Design waivers will be required for shared use paths that do not meet the minimum requirements in the AASHTO Guide for the Development of Bicycle Facilities.
Chapter 7 — Miscellaneous Design Elements

Contents:

Section 1 — Longitudinal Barriers and Roadside Safety Hardware Criteria
Section 2 — Fencing
Section 3 — Pedestrian Separations and Ramps
Section 4 — Parking
Section 5 — Rumble Strips
Section 6 — Emergency Median Openings on Freeways
Section 7 — Minimum Designs for Truck and Bus Turns
Section 1 — Longitudinal Barriers and Roadside Safety Hardware Criteria

Overview

This section contains information regarding the following elements of longitudinal barriers:

- Concrete Barriers (Median and Roadside),
- Guardrail, and
- Attenuators (Crash Cushions),
- Roadside Safety Hardware Criteria.

Concrete Barriers (Median and Roadside)

Application. Concrete barriers may be used to prevent the following:

- Unlawful turns
- Out-of-control vehicles from entering the opposing traffic lanes, and, in some cases
- Unlawful crossing of medians by pedestrians.

Concrete barriers, much like guardrail, may also be used as roadside barriers to prevent vehicles from encountering steep slopes or obstacles.

Location. On controlled access highways, concrete barriers will generally be provided in medians of 30 ft [9.0 m] or less. On non-controlled access highways, concrete barriers may be used on medians of 30 ft [9.0 m] or less; however, care should be exercised in their use in order to avoid the creation of an obstacle or restriction in sight distance at median openings or on horizontal curves. Generally, the use of concrete barriers on non-controlled access facilities should be restricted to areas with potential safety concerns such as railroad separations or through areas where median constriction occurs. Concrete barriers may be considered in medians wider than 30 ft [9.0 m] based on an operational analysis.

Standard Installations. Medians for urban freeway sections generally are relatively narrow and flush. For new construction, an urban freeway usually includes a flush median (see Medians in Chapter 3) with concrete barrier.

In determining the type of barrier to be used for any project, the primary consideration is safety, both for vehicular impacts and during any maintenance activities. Field experience with concrete barriers indicates that, unlike the metal beam system, maintenance operations are not normally required following accidental vehicular encroachment.
Reconstruction projects with median barriers should be considered on a project-by-project basis. Often, the structural capability of existing bridges may make the use of concrete median barriers infeasible due to increased dead load.

TxDOT’s design standards and standard construction specifications provide more information on the design and construction details for concrete barriers.

Guardrail

**Application.** Guardrail is considered a protective device for the traveling public and is used at points on the highway where vehicles inadvertently leaving the facility would be a significant safety concern. Guardrail is designed to resist impact by deflecting the vehicle so that it continues to move at a reduced velocity along the rail in the original direction of traffic. The limits of rail to be installed are shown on the plans; however, they may be adjusted in the field after the grading is completed.

**Location.** Guardrail should be installed in areas where the consequence of an errant vehicle leaving the roadway is judged to be more severe than impacting the guardrail. Guardrail should be offset at least 4 ft [1372 mm] and desirably 5 ft [1,500 mm] or more from the nearest edge of fixed objects. At overpasses, guardrail should be anchored securely to the structure.

**Standard Installations.** Guardrail should be installed in accordance with the current roadway standards.

**End Treatments.** Providing appropriate end treatments is one of the most important considerations in the design of guardrail. An untreated guardrail will stop a vehicle abruptly and can penetrate the passenger compartment. For more information on the installation of various types of end treatments, refer to TxDOT’s standard construction specifications and roadway standards.

Attenuators (Crash Cushions)

**Application.** Crash cushions or impact attenuators are protective devices that prevent errant vehicles from impacting fixed objects. This is accomplished by gradually decelerating a vehicle to a safe stop for head-on impacts or redirecting a vehicle away from the fixed object for side impacts.

**Location.** Attenuators are ideally suited for use at locations where fixed objects cannot be moved, relocated, or made breakaway, and cannot be adequately shielded by a longitudinal barrier. A common application of a crash cushion is in an exit ramp gore where a bridge rail end requires shielding. Crash cushions are also frequently used to shield bridge columns as well as roadside and median barrier terminals.

**Standard Installations.** There are numerous types of attenuators that are in common use today. When more than one system is under consideration, the designer should carefully evaluate the
structural, safety, and maintenance characteristics of each candidate system. Characteristics to be considered include the following:

◆ Impact decelerations,
◆ Redirection capabilities,
◆ Anchorage and back-up structure requirements,
◆ Debris produced by impact, and
◆ Ease and cost of maintenance.

Crash Cushion Categories. Crash cushions are classified in one of three categories based on the reusability of the product after a head on impact: Sacrificial, Reusable, and Low Maintenance.

Sacrificial: These units are typically filled with water or sand and typically require full replacement or substantial repairs either on-site or in a maintenance yard following an impact. These units have low initial costs and should be considered for sites that typically experience less than one impact every 18 months. Water-filled crash cushions are allowed for use in temporary work zones only.

Reusable: These units typically feature side fender panels, cartridges, or cylinders that absorb an errant vehicle's energy during impact. Typically, the cartridges, or cylinders and a nose piece will have to be replaced after an impact. These units should be considered in locations that typically experience impacts in the range between one impact every 18 months and less than 3 impacts per year.

Low Maintenance: These units typically utilize plastic cylinders, or hydraulic mechanisms to absorb energy. These require some maintenance after an impact to ensure proper performance during the next impact. These have high initial cost and should typically be considered at locations where they will experience 3 or more impacts per year.

For more detailed information on the installation of various types of attenuators, refer to TxDOT’s standard specifications and roadway standards.
Roadside Safety Hardware Criteria


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 (2420 lbs.) and a large pick-up (5000 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 22,000 lb. delivery type truck. The primary difference between MASH and the earlier NCHRP 350 criteria is the increase in size and height of the tested pick-up truck to account the change in vehicle fleet, and to better simulate an SUV. Other changes include the small car weight, and angle of impact.

MASH Background: On November 20, 2009, a memorandum from David A. Nicol was issued about the “AASHTO Manual for Assessing Safety Hardware (MASH).” This AASHTO manual supersedes NCHRP Report 350 for the purposes of evaluating new safety hardware such as longitudinal barriers, transitions, end terminals, crash cushions, breakaway/yielding supports, truck mounted attenuators, and work zone traffic control devices. It sets guidelines for crash testing and evaluation criteria for assessing test results. The joint AASHTO/FHWA implementation plan states that all highway safety hardware accepted under the criteria in NCHRP Report 350 does not need to be retested to MASH criteria; may remain in place; and may continue to be manufactured and installed. However, all new hardware that is developed must be tested and evaluated according to MASH.

◆ May 21, 2012, memorandum from Tony Furst on the subject of Roadside Safety Hardware - Federal-Aid Reimbursement Eligibility Process and related Frequently Asked Questions. Establishes that States can certify that roadside safety hardware has been tested by an accredited crash test laboratory and meets MASH criteria, and can thus be eligible for reimbursement.

◆ January 7, 2016 memorandum from Thomas Everett on the subject of “AASHTO/FHWA Joint Implementation Agreement for Manual for Assessing Safety Hardware (MASH).” The memo discusses the agreement between AASHTO and FHWA that requires all new installations of safety hardware on the NHS to be evaluated using the 2016 edition of MASH. The requirement applies to bridge railings with contract letting dates after December 31, 2019.

Current TxDOT MASH Implementation Timetable/Policy:

As product manufacturers and developers have tried to develop MASH 2016 compliant products, the FHWA in coordination with AASHTO has allowed additional flexibility with respect to the
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 Guardrail End Treatments 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 those standards (whether MASH 2016, MASH 2009, or NCHRP 350) that are available for use until further notice.

Bridge rails, transitions, all other longitudinal barrier (including portable barrier 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 those categories of devices where a MASH 2016 alternative may not be available. Note that all current Bridge Railing standards (BRG), permanent sign support standards (TRF), and Mailbox standards (MNT) are MASH 2016 compliant.

Temporary work zone devices (including portable barriers and water-filled crash cushions): For these devices, those 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 currently 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.

The end result for these various categories of roadside safety hardware items is that for TxDOT all of the standards available on the respective TxDOT Division standards webpage are available for use until future notice. Over time, 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 Design Division's (Roadway and Hydraulic Design Section) Webpage.
Section 2 — Fencing

Right-of-way

The procedures for fencing highway right-of-way are in the ROW Acquisition Manual. Where additional right-of-way is not required for construction of improvements of existing highways, right-of-way (property) fencing is the responsibility of the land owner.

Control of Access Fencing on Freeways

Control of access fence should be erected whenever it is necessary to prohibit unrestricted access to the through lanes by pedestrians, animals and/or vehicles. The prohibition of access to the through lanes should be from private property, intercepted local roads and unauthorized crossings from frontage roads to the through lanes. Table 7-1 describes the types of fences that should be used for various conditions.

Department standard designs should be used where applicable. Specially designed fences may be necessary in certain areas where sandstorms and snowstorms occur and for other special conditions.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Type of Fence</th>
<th>Usual Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Urban and suburban areas</td>
<td>Chain link fence of 4 ft [1.2 m] usual height or 6 ft [1.8 m] height where necessary for control of pedestrians</td>
<td>Variable¹</td>
</tr>
<tr>
<td>Rural conditions where both large and small animals exist</td>
<td>Wire mesh fence with one or more strands of barbed wire</td>
<td>ROW line</td>
</tr>
<tr>
<td>Rural conditions where only large animals exist</td>
<td>Barbed wire fence with height of 4 ft or 5 ft [1.2 or 1.5 m]</td>
<td>ROW line</td>
</tr>
<tr>
<td>Control of Vehicles</td>
<td>Post and cable fence with closely spaced posts</td>
<td>Variable¹</td>
</tr>
</tbody>
</table>

¹Where frontage roads are provided, control of access fence, when used, should be placed in the outer separation approximately equidistant between the mainlanes and frontage roads and at least 30 ft [9.0 m] from the edge of mainlane pavement. Where the control of access line is at the right-of-way line, the control of access fence may be located at the right-of-way line and will serve a dual function as a right-of-way fence.
Section 3 — Pedestrian Separations and Ramps

General Requirements

Pedestrian separations are generally limited to controlled access facilities since it is necessary that all at-grade pedestrian crossings be eliminated on those facilities. Control-of-access fences and other means may be used to encourage pedestrians to cross at traffic separations. On highways other than freeways, pedestrian separations will be considered only in unusual circumstances.

Pedestrian structures may be used to provide for heavy pedestrian movements adjacent to factories, schools, parks, athletic fields, etc. If the location of traffic separations is such that their use would add an unreasonable pedestrian distance, a pedestrian structure may be considered for lower pedestrian volumes.

A pedestrian structure should be made as natural and convenient as possible. Either an overcrossing or undercrossing may be provided. All separations must be accessible to the disabled unless alternate safe means are provided to enable mobility-limited persons to cross the roadway at that location, or unless it would be infeasible for mobility-limited persons to reach the structure because of unusual topographical or architectural obstacles unrelated to the roadway facility.

Pedestrian ramps associated with roadway facilities such as pedestrian separations, parking lots, rest areas, curb cuts at cross walks, etc., must be accessible to disabled persons and designed in accordance with the Americans with Disabilities Act Accessibility Guidelines and the Texas Accessibility Standards.

Overcrossings

All pedestrian overcrossings should be enclosed with wire fabric to discourage pedestrians from throwing debris onto vehicles below the structure.

Undercrossings

Pedestrians avoid the use of undercrossings unless the underpass is in line with the approach sidewalk and has continuous vision through the underpass from the approaching sidewalk. Ample lighting and drainage, both day and night, is essential.
Section 4 — Parking

Overview

This section presents information on the following topics:

- Fringe parking lots,
- Parking along highways and arterial streets

Fringe Parking Lots

Fringe parking lots are congestion mitigation and energy conservation measures which TxDOT utilizes. Depending on the function which they are intended to serve, they maybe one of the following types of facilities:

- Park and pool lots,
- Park and ride lots, or, and
- Combination park and pool/park and ride lot.

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 vehicles. The carpool may consist of two or more persons per vehicle. The lot may provide space for a small to large number of vehicles and serve many carpools involving several destinations.

Park and pool lots are located within the highway right-of-way except where they may be in combination with a park and ride lot as discussed below. They are eligible for Federal-aid participation. The lots should be simply designed to accommodate the passenger vehicle with regard to parking stall widths, drive through isles and turning movements.

Park and Ride Lots. Park and ride lots are generally constructed along express bus routes and are designed to intercept automobiles from low density suburban development of outlying locations 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 car.

The facility should be located with regard to the following criteria:

- Along a corridor which experiences 20,000 vehicles per day per lane,
- In advance of the point where intense traffic congestion routinely occurs,
4 to 5 miles [6.4 to 8.0 kilometers] from the activity center (usually the Central Business District (CBD)) served by the transit way and at least 4 to 5 miles [6.4 to 8.0 kilometers] 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 general 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 Bus or A-Bus for all turning movements,
- Bus loading areas located to reduce conflict between buses and private vehicles,
- Maximum walking distance of 650 ft [200 m],
- Separate bus access points from private vehicle access points if demand exceeds 500 all day spaces,
- Parking placed in the following order with respect to proximity of the bus loading area:
  - Disabled persons,
  - Bicycles,
  - Motorcycles,
  - Park and ride, and
  - Private vehicular parking.
- Ingress and egress located near midblock on collector and local streets; direct access to arterials and freeway ramps should only be used if projected queues do not interfere with functional areas of nearby intersections; at least two ingress/egress points should be provided to the park and ride facility; right and left turn lanes with adequate storage should be added to all ingress/egress locations,
- Parking lanes in the park and ride lot placed approximately 90 degrees to the bus loading area to facilitate safe, convenient walking to buses, and
- Curbs depressed and wheelchair ramps provided where necessary; disabled parking spaces and pedestrian facilities should be in accordance with the Americans with Disabilities Act Accessibility Guidelines and the Texas Accessibility Standards.
Combination Park and Pool/Park and Ride Lot. These combination type lots serve the purposes and combine the features of each of the two types of facilities discussed above.

References. Further information on the planning and design of park and ride facilities may be found in the following publications:

- AASHTO Policy on Geometric Design of Streets and Highways
- AASHTO Guide for the Design of Park and Ride Facilities
- TxDOT Revised Manual for Planning, Designing and Operating Transitway Facilities in Texas.

Authority and Funding. For fringe parking areas within highway right-of-way, projects are generally developed as any other multiple use project. Where parking lots are proposed that are located outside of existing or proposed highway right-of-way, commission approval is required.

Park and pool lots are eligible for Federal-aid participation. Projects are usually located within or adjacent to highway right-of-way outside the central business district, but inside the urbanized area, and consistent with the urban transportation planning process. Operation and maintenance responsibilities should be assigned to local transit or government or agencies by agreement.

Parking Along Highways and Arterial Streets

This section deals with parking as it pertains to the mainlanes of a controlled access highway, the frontage roads for such a facility, and parking along urban and suburban arterials. Off-street parking facilities provided within highway right-of-way are discussed in the previous section (Fringe Parking Lots) Rest areas as parking facilities are not considered in this section.

Emergency Parking. Parking on and adjacent to the mainlanes of a highway will not be permitted except for emergency situations. It is of paramount importance, however, that provision be made for emergency parking. Shoulders of adequate design provide for this required parking space.

Curb Parking. In general, curb parking on urban/suburban arterial streets and frontage roads should be discouraged. Where speed is low and the traffic volumes are well below capacity, curb parking may be permitted. However, at higher speeds and during periods of heavy traffic movement, curb parking is incompatible with arterial street service and desirably should not be permitted. Curb parking reduces capacity and interferes with free flow of adjacent traffic. Elimination of curb parking can increase the capacity of four-to-six lane arterials by 50 to 60 percent.

If curb parking is used on urban/suburban arterials or frontage roads under the conditions stated above, the following design requirements should be met:

- Provide parking lanes only at locations where needed,
- Parallel parking preferred,
Confine parking lanes to outer side of street or frontage road,

- Require that parking lane widths be 10 feet [3.0 meters], and

- Restrict parking a minimum of 20 feet [6 meters] back from the radius of the intersection to allow for sight distance, turning clearance and, if desired, a short right turn lane.
Section 5 — Rumble Strips

Overview

Centerline and shoulder rumble strips are depressed or raised patterns used to provide auditory and tactile sensations to the driver as a warning mechanism when they leave the travel way. 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 the use of these rumble strips are specified in the respective Traffic Safety Division RS standard. Rumble strips have been shown to be a cost-effective countermeasure for reducing the number and severity of roadway departure crashes. As such, rumble strips have been incorporated into the Safety Score Tools developed by TxDOT’s Council on System Safety. Additional information on the Safety Score Tools is available on the TxDOT Design Division webpage.

Considerations for Centerline and Shoulder Rumble Strip Placement: Rumble strips shall not be placed on roadways with a posted speed limit of 45 mph or less. For rural high-speed roadways, rumble strips should be installed as part of new construction, reconstruction and overlay projects, unless engineering/safety judgment determine it would be detrimental to do so. Rumble strips are recommended on high-speed urban roadways where significant numbers of crashes, by frequency and percentage of total accidents, that involve any form of motorist inattention have been identified (e.g., opposing direction crashes, run-off-road crashes). Shoulder rumble strips shall not be placed across exit or entrance ramps, acceleration and deceleration lanes, crossovers, gore areas or intersections with other roadways. Depressed rumble strips shall not be placed across bridge decks. If a concrete shoulder will be used in the near future as a permanent travel lane or a travel lane in a work zone, depressed rumble strips should not be used.

Bicyclists considerations: In all installations, appropriate riding space for bicyclists should be a consideration. It is preferred to allow at least four feet beyond the rumble strips to the edge of the paved shoulder. On some facilities known to have considerable bicycle traffic, providing occasional gaps should be considered to allow bicyclists to traverse in and out of the shoulder safely.

See Traffic Safety Division’s Rumble Strip standards (RS standards), and FHWA rumble strip guidance for additional information:
https://safety.fhwa.dot.gov/roadway_dept/pavement/rumble_strips/
Section 6 — Emergency Median Openings on Freeways

Overview

Median crossings between the mainlanes are sometimes necessary for proper law enforcement or for performance of highway maintenance on rural freeways. The construction of such median crossings is not encouraged since the necessary U-turns by such vehicles should be accomplished by using ramps at interchanges to the maximum extent feasible.

Conditions

Median crossings, as turnarounds, interfere with through traffic and should be avoided. Normally, the spacing of interchanges and layout of the highway provides for all necessary traffic movements, including those of emergency vehicles.

In unusual situations, where the distance between interchanges is great, emergency crossings may be provided with administrative approval.

Spacing of Openings

Due to the close spacing of interchanges on urban freeways, emergency median openings are not needed for the operation of official vehicles and, in general, they should not be provided. In rural areas where the spacing of interchanges is greater than approximately 3 miles [4.8 km], a U-turn median opening may be considered at a favorable location about halfway between interchanges. In no case should emergency median openings be spaced at less than 1 mile [1.6 km] intervals. All emergency median openings should be at least 0.5 mile [0.8 km] from any structure that crosses over a freeway and at least 1 mile [1.6 km] from any ramp terminal or other access connection, such as those serving safety rest areas. Openings 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 freeway lanes. Emergency median openings should also be as inconspicuous to the traveling public as possible.

Construction

Location and type of emergency median openings should be made a part of the PS&E as a contract item and should be installed as such.
Section 7 — Minimum Designs for Truck and Bus Turns

Overview

This section contains the following information on minimum designs for truck and bus turns:

- Application,
- Channelization,
- Alternatives to simple curvature,
- Urban intersections, and
- Rural intersections.

Application

There are no firm guidelines governing the selection of the type of large vehicle to be used as a design vehicle. Factors that influence design vehicle selection are as follows:

- Type and frequency of use by large vehicles,
- Consequences of encroachment into other lanes or the roadside,
- Availability of right-of-way, and
- Functional class of intersecting routes and location (urban versus rural) affect this selection in a general sense. Project-specific traffic data, specifically the frequency of use by the various design vehicle classes, is often the most important consideration in the selection process. The Transportation Planning and Programming Division (TPP) may be contacted to obtain volume data for the various vehicle classes.

Minimum turning path templates for single unit trucks or buses, semi-trailer combinations with wheelbases of 40, 50 and 62 ft [12.2, 15.24 and 18.9 m], and double-trailer combination with wheelbase of 67 ft [20.43 m] are shown in Figures 7-1, 7-2, 7-3, 7-4, 7-5, and 7-6 respectively. The AASHTO publication A Policy on Geometric Design of Highways and Streets provides additional information on turning paths and turning radii of these and other vehicles.
Figure 7-1. Turning Template for Single Unit Trucks or Buses, (not to scale).

NOTE: According to AASHTO's Policy on Geometric Design of Highways and Streets (2018), the 'SU' design accommodates the inside turning radii for the six types of buses and all but one (BUS-45, intercity) of the outside turning radii. If bicycle racks are a consideration for buses, see AASHTO for additional outside turning radii requirements.
Figure 7-2. Turning Template for Semi-Trailer with 40 ft [12.20 m] Wheelbase, (not to scale).
Figure 7-3. Turning Template for Semi-Trailer with 50 ft [15.24 m] Wheelbase, (not to scale).
Figure 7-4. Turning Template for Semi-Trailer with 62 ft [18.9 m] Wheelbase, (not to scale).
Figure 7-5. Turning Template for Semi-Trailer with 62 ft [18.9 m] Wheelbase (Radius = 75 ft [22.9 m], (not to scale).
Figure 7-6. Turning Template for Double-Trailer Combination with 67 ft [20.41 m] Wheelbase, (figure not to scale).
Figure 7-7. (US). Example of Pavement Edge Geometry (US Customary).
Channelization

Where the inner edges of pavement for right turns at intersections are designed to accommodate semi-trailer combinations or where the design permits passenger vehicles to turn at 15 mph [20 km/h] or more (i.e., 50 ft [15 m] or more radius), the pavement area at the intersection may become excessively large for proper control of traffic. In these cases, channelizing islands should be used to more effectively control, direct, and/or divide traffic paths. Physically, islands should be at least 50 ft² [4.5 m²] in urban and 75 ft² [7.0 m²] for rural conditions (100 ft² [9.0 m²] preferable for both) in size and may range from a painted to a curbed area.
Alternatives to Simple Curvature

To accommodate the longest vehicles, off-tracking characteristics in combination with the large (simple curve) radius that must be used results in a wide pavement area. In this regard, three-centered compound curves, or offset simple curves in combination with tapers, are preferred since they more closely fit the paths of vehicles. Table 7-2 shows minimum edge of pavement designs for right turns to accommodate various design vehicles for turn angles varying from 60 to 120 degrees.

Table 7-2: Minimum Edge of Pavement Designs for Right Turns for Various Design Vehicles for Turn Angle Varying from 60 to 120 Degrees

<table>
<thead>
<tr>
<th>Angle of Turn (degrees)</th>
<th>Design Vehicle</th>
<th>Simple Curve Radius (ft.)</th>
<th>Simple Curve Radius with Taper</th>
<th>3-Centered Compound Curve, Symmetric</th>
<th>3-Centered Compound Curve, Asymmetric</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>(US Customary)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>P</td>
<td>40</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>SU</td>
<td>60</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>WB-40</td>
<td>90</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>WB-50</td>
<td>150</td>
<td>120</td>
<td>3.0</td>
<td>15:1</td>
</tr>
<tr>
<td>75</td>
<td>P</td>
<td>35</td>
<td>25</td>
<td>2.0</td>
<td>10:1</td>
</tr>
<tr>
<td>-</td>
<td>SU</td>
<td>55</td>
<td>45</td>
<td>2.0</td>
<td>10:1</td>
</tr>
<tr>
<td>-</td>
<td>WB-40</td>
<td>-</td>
<td>60</td>
<td>2.0</td>
<td>15:1</td>
</tr>
<tr>
<td>-</td>
<td>WB-50</td>
<td>-</td>
<td>65</td>
<td>3.0</td>
<td>15:1</td>
</tr>
<tr>
<td>90</td>
<td>P</td>
<td>30</td>
<td>20</td>
<td>2.5</td>
<td>10:1</td>
</tr>
<tr>
<td>-</td>
<td>SU</td>
<td>50</td>
<td>40</td>
<td>2.0</td>
<td>10:1</td>
</tr>
<tr>
<td>-</td>
<td>WB-40</td>
<td>-</td>
<td>45</td>
<td>4.0</td>
<td>10:1</td>
</tr>
<tr>
<td>-</td>
<td>WB-50</td>
<td>-</td>
<td>60</td>
<td>4.0</td>
<td>15:1</td>
</tr>
<tr>
<td>105</td>
<td>P</td>
<td>-</td>
<td>20</td>
<td>2.5</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>SU</td>
<td>-</td>
<td>35</td>
<td>3.0</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>WB-40</td>
<td>-</td>
<td>40</td>
<td>4.0</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>WB-50</td>
<td>-</td>
<td>55</td>
<td>4.0</td>
<td>15:1</td>
</tr>
<tr>
<td>120</td>
<td>P</td>
<td>-</td>
<td>20</td>
<td>2.0</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>SU</td>
<td>-</td>
<td>30</td>
<td>3.0</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>WB-40</td>
<td>-</td>
<td>35</td>
<td>5.0</td>
<td>-</td>
</tr>
</tbody>
</table>
"Angle of Turn" is the angle through which a vehicle travels in making a turn. It is measured from the extension of the tangent on which a vehicle approaches to the corresponding tangent on the intersecting road to which a vehicle turns. It is the same angle that is commonly called the delta angle in surveying terminology.

<table>
<thead>
<tr>
<th>Angle of Turn (degrees)</th>
<th>Design Vehicle</th>
<th>Simple Curve Radius (ft.)</th>
<th>Simple Curve Radius with Taper</th>
<th>3-Centered Compound Curve, Symmetric</th>
<th>3-Centered Compound Curve, Asymmetric</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Radius (ft.)</td>
<td>Offset (ft.)</td>
<td>Taper</td>
<td>Radii (ft.)</td>
</tr>
<tr>
<td>- WB-50</td>
<td></td>
<td>45</td>
<td>4.0</td>
<td>15:1</td>
<td>180-40-180</td>
</tr>
</tbody>
</table>

1 “Angle of Turn” is the angle through which a vehicle travels in making a turn. It is measured from the extension of the tangent on which a vehicle approaches to the corresponding tangent on the intersecting road to which a vehicle turns. It is the same angle that is commonly called the delta angle in surveying terminology.
Figure 7-7 shows sample alternative (to simple curvature) edge of pavement geometry for a 90 degree turn using a WB 50 [WB-15] design vehicle. Although not shown in this figure, a radius of 80 ft [25 m] without channelizing island would be necessary to accommodate the wide, off-tracking path of a WB-50 [WB-15] without undesirable encroachment. A geometric design of this sort is undesirable, however, since there would be a confusing, wide expanse of surfaced area; furthermore, there is no convenient, effective location for traffic control devices.

### Urban Intersections

Corner radii at intersections on arterial streets should satisfy the requirements of the drivers using them to the extent practical and in consideration of the amount of right-of-way available, the angle of the intersection, numbers of and space for pedestrians, width and number of lanes on the intersecting streets, and amounts of speed reductions. The following summary is offered as a guide:

- Radii of 15 ft [4.5 m] to 25 ft [7.5 m] 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 [7.5 m] or more at minor cross streets should be provided on new construction and on reconstruction where space permits.

### Table 7-2: Minimum Edge of Pavement Designs for Right Turns for Various Design Vehicles for Turn Angle Varying from 60 to 120 Degrees

<table>
<thead>
<tr>
<th>Angle of Turn (degrees)</th>
<th>Design Vehicle</th>
<th>Simple Curve Radius (m)</th>
<th>Simple Curve Radius with Taper</th>
<th>3-Centered Compound Curve, Symmetric</th>
<th>3-Centered Compound Curve, Asymmetric</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>P</td>
<td>12</td>
<td>1.2</td>
<td>10:1</td>
<td>30-11-30</td>
</tr>
<tr>
<td>-</td>
<td>WB-12</td>
<td>-</td>
<td>17</td>
<td>1.2</td>
<td>15:1</td>
</tr>
<tr>
<td>-</td>
<td>WB-15</td>
<td>-</td>
<td>6</td>
<td>0.6</td>
<td>10:1</td>
</tr>
<tr>
<td>-</td>
<td>SU</td>
<td>-</td>
<td>9</td>
<td>1.0</td>
<td>10:1</td>
</tr>
<tr>
<td>-</td>
<td>WB-12</td>
<td>-</td>
<td>11</td>
<td>1.5</td>
<td>8:1</td>
</tr>
<tr>
<td>-</td>
<td>WB-15</td>
<td>-</td>
<td>14</td>
<td>1.2</td>
<td>15:1</td>
</tr>
</tbody>
</table>

1°Angle of Turn” is the angle through which a vehicle travels in making a turn. It is measured from the extension of the tangent on which a vehicle approaches to the corresponding tangent on the intersecting road to which a vehicle turns. It is the same angle that is commonly called the delta angle in surveying terminology.
 Radii of 30 ft [9 m] 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 [12 m] 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 dimensions should be coordinated with crosswalk distances or special designs to make crosswalks safe for all pedestrians.

For arterial-arterial urban intersections, turning radii of 75 ft [23 m] or more are desirable if frequent use is anticipated by the WB-62 [WB-19] design vehicle. Where other types of truck combinations are used as the design vehicle, pavement edge geometry as shown in Table 7-2: Minimum Edge of Pavement Designs at Intersections and Figure 7-7 permit the use of lesser radii. An operational measure that appears promising is to provide guidance in the form of edge lines to accommodate the turning paths of passenger cars, while providing sufficient paved area beyond the edge lines to accommodate the turning path of an occasional large vehicle.

Additional Right-Turn slip lane guidance is provided in Appendix D.

**Rural Intersections**

In rural areas space is generally more available and speeds higher. These factors suggest more liberal designs for truck turning even when the frequency of long vehicles may not be as great as in urban areas.

In the design of highway intersections with other (non-highway system) public roads, long vehicles are generally infrequent users. Minimally, the SU, or on some occasions the WB –40 [WB-12], 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 [WB-12] design vehicle is generally appropriate and the WB-50 [WB-15] should be used where specific circumstances warrant.

For arterial-arterial intersections, use by the WB-62 [WB-19] design vehicle should be anticipated within project life. Two template layouts, Figure 7-4 and Figure 7-5, are shown with radii of 45 ft [13.7 m] and 75 ft [23 m] respectively. For turning roadway widths to be reasonable in width, a design radius of 75 ft [23 m] or more is required. Where circumstances at a particular rural arterial-arterial intersection precludes the use of the WB-62 [WB-19] design vehicle, the WB-50 [WB-15] should be used.
Chapter 8 — Mobility Corridor (5 R) Design Criteria

Contents:

Section 1 — Overview
Section 2 — Roadway Design Criteria
Section 3 — roadside Design Criteria
Section 4 — Ramps and Direct Connections
Chapter 8 — Mobility Corridor (5 R) Design Criteria

Section 1 — Overview

Introduction

Mobility corridors are intended to generate, or produce anew, very 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 additional experience is gained in the planning, design, construction and operations of these transportation facilities.

The primary focus of these corridors is mobility. The roadway portions of a mobility corridor facility are intended for long distance travel, and will therefore, be very controlled in terms of access. The access will be limited to public roadways via ramp connections. Access will not be allowed along these ramp connections.

Since these corridors are intended for mobility, the design speeds presented in this chapter are between 85 mph to 100 mph [130 to 160 km/h]. Because mobility corridors may be generated or regenerated, this design criteria may be applied when planning new facilities or reconstructing existing corridors. While higher operating speeds may not be appropriate in all instances (such as densely developed urban areas), these higher design speeds can be applied, and should be considered, whenever prudent.

With respect to facilities that one day could be part of a major corridor, particularly new location routes, it is strongly recommended that these facilities be initially designed to accommodate a 100 mph design speed. Even though the facility may initially be posted for an 85 mph speed, the higher design criteria will allow the greatest flexibility, both in the roadway portion as well as for other transportation modes within the right of way, in terms of maximizing the future use of the corridor.

This does not mean that all projects should be over-designed. If, through the project development process it is determined that substantial, adverse and unavoidable social, economic and environmental impacts will occur, then different design criteria may be appropriate. Contact the Environmental Affairs Division and the Right of Way Division as questions arise about environmental and right of way impacts while planning for higher design speeds.

As always, the potential long-term use and growth of the system should be considered and appropriate planning and engineering principles should be applied. Again, these mobility corridors are not primarily intended for local travel.

Section 2 discusses the features and design criteria for the roadway portion of mobility corridors and includes the following subsections.
Section 1 — Overview

- Lane Width and Number
- Shoulders
- Pavement Cross Slope
- Vertical Clearances at Structures
- Stopping Sight Distance
- Grades
- Curve Radii
- Superelevation
- Vertical Curves

Departures from these guidelines are governed in Design Exceptions, Design Waivers, Design Variations, and Texas Highway Freight Network (THFN) Design Deviations, Chapter 1.
Section 2 — Roadway Design Criteria

Lane Width and Number

The usual and minimum lane width is 13 ft [4 m]. The number of lanes required to accommodate the anticipated traffic in the design year is determined by the level of service evaluation as discussed in the Highway Capacity Manual.

Shoulders

The minimum shoulder width is 12 ft [3.6 m]. This width applies to both inside and outside shoulders, regardless of the number of main lanes of the facility. Shoulders should be continuously surfaced and be maintained along all speed change lanes.

Pavement Cross Slope

Multilane divided pavements should be inclined in the same direction. The recommended pavement cross slope is 2.0 percent. Shoulders should be sloped sufficiently to drain surface water but not to an extent that safety concerns are created for vehicular use.

Vertical Clearances at Structures

The minimum vertical clearances at structures for the facility types are as described below:

- Texas Highway Freight Network (THFN) as specified in Chapter 3, Section 8.
- Other Facilities as specified in Chapter 3, Section 6.

Stopping Sight Distance

Stopping sight distance (SSD) for these facilities is calculated using the same methodology described in Chapter 2, Section 3. The key variables that affect the calculation of SSD are brake reaction time and deceleration rate. The calculated and design stopping sight distances are shown in Table 8-1. Significant downgrades may affect stopping sight distances.
Grades

Undesirable speed differentials that could occur between vehicle types on these facilities suggest that limiting the rate and length of the grades be considered. Passenger vehicles are not significantly affected by grades as steep as three percent, regardless of initial speed. Grades above two percent may affect truck traffic depending on length of grade.

Table 8-2 summarizes the maximum grade controls in terms of design speed.

### Table 8-1: Stopping Sight Distance

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Brake reaction distance (ft)</th>
<th>Braking distance on level (ft)</th>
<th>Stopping Sight Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Calculated (ft)</td>
</tr>
<tr>
<td>85</td>
<td>312.4</td>
<td>693.5</td>
<td>1005.8</td>
</tr>
<tr>
<td>90</td>
<td>330.8</td>
<td>777.5</td>
<td>1108.2</td>
</tr>
<tr>
<td>95</td>
<td>349.1</td>
<td>866.2</td>
<td>1215.4</td>
</tr>
<tr>
<td>100</td>
<td>367.5</td>
<td>959.8</td>
<td>1327.3</td>
</tr>
</tbody>
</table>

### Table 8-2: Maximum Grades

<table>
<thead>
<tr>
<th>Type of Terrain</th>
<th>Design Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>85</td>
</tr>
<tr>
<td>Level</td>
<td>2-3</td>
</tr>
<tr>
<td>Rolling</td>
<td>4</td>
</tr>
</tbody>
</table>
Curve Radii

The minimum curve radii for superelevation rates of 6 percent and 8 percent are shown in Table 8-3. These radii were calculated using the horizontal curvature equation shown in Chapter 2, section 4, with the side friction values in Table 8-5 and the assumed maximum superelevation rates.

Table 8-3: Horizontal Curvature Highways and Connecting Roadways with Superelevation

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Usual Min. Radius of Curve (ft)</th>
<th>Absolute Min.¹ Radius of Curve (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>5615</td>
<td>3710</td>
</tr>
<tr>
<td>90</td>
<td>6820</td>
<td>4500</td>
</tr>
<tr>
<td>95</td>
<td>8285</td>
<td>5470</td>
</tr>
<tr>
<td>100</td>
<td>10100</td>
<td>6670</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Speed (km/h)</th>
<th>Usual Min. Radius of Curve (m)</th>
<th>Absolute Min.¹ Radius of Curve (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>140</td>
<td>1800</td>
<td>1190</td>
</tr>
<tr>
<td>150</td>
<td>2440</td>
<td>1615</td>
</tr>
<tr>
<td>160</td>
<td>3050</td>
<td>2020</td>
</tr>
</tbody>
</table>

¹ Absolute minimum values should be used only where unusual design circumstances dictate.
Horizontal curvature without superelevation means maintaining a normal crown with a negative 2 percent superelevation for one direction, and the side friction is not excessive for that direction. Table 8-5 shows the minimum curve radii without additional superelevation and an e_{\text{max}} of 8 percent.
Table 8-5: Horizontal Curvature of Highways without Superelevation

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Min. Radius (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>14700</td>
</tr>
<tr>
<td>90</td>
<td>16200</td>
</tr>
<tr>
<td>95</td>
<td>18800</td>
</tr>
<tr>
<td>100</td>
<td>22400</td>
</tr>
</tbody>
</table>

(Metric)

<table>
<thead>
<tr>
<th>Design Speed (km/h)</th>
<th>Min. Radius (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>140</td>
<td>4680</td>
</tr>
<tr>
<td>150</td>
<td>5480</td>
</tr>
<tr>
<td>160</td>
<td>6750</td>
</tr>
</tbody>
</table>

1Normal crown (2%) maintained ($e_{\text{max}} = 8\%$)

Superelevation

The maximum superelevation rates of 6 to 8 percent are not varied based on design speed.

Tables 8-6 and 8-7 show superelevation rates (maximum 6 and 8 percent, respectively) for various design speeds and radii.

Table 8-6: Superelevation Rates for Horizontal Curves:
Superelevation Rate, $e$ (6%), for Design Speed of
(U.S. Customary)

<table>
<thead>
<tr>
<th>Radius (ft.)</th>
<th>85 mph</th>
<th>90 mph</th>
<th>95 mph</th>
<th>100 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>23000</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
</tr>
<tr>
<td>20000</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
<td>2.2</td>
</tr>
<tr>
<td>17000</td>
<td>NC</td>
<td>NC</td>
<td>2.2</td>
<td>2.6</td>
</tr>
<tr>
<td>14000</td>
<td>RC</td>
<td>2.3</td>
<td>2.6</td>
<td>3.2</td>
</tr>
<tr>
<td>12000</td>
<td>2.4</td>
<td>2.6</td>
<td>3.0</td>
<td>3.6</td>
</tr>
<tr>
<td>10000</td>
<td>2.8</td>
<td>3.1</td>
<td>3.6</td>
<td>4.3</td>
</tr>
<tr>
<td>8000</td>
<td>3.4</td>
<td>3.8</td>
<td>4.5</td>
<td>5.3</td>
</tr>
<tr>
<td>6000</td>
<td>4.5</td>
<td>5.0</td>
<td>5.8</td>
<td>$R_{\text{min}} = 6670$ ft</td>
</tr>
</tbody>
</table>
### Table 8-6: Superelevation Rates for Horizontal Curves:
Superelevation Rate, $e$ (6%), for Design Speed of
(US Customary)

<table>
<thead>
<tr>
<th>Radius (ft.)</th>
<th>85 mph</th>
<th>90 mph</th>
<th>95 mph</th>
<th>100 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>5000</td>
<td>5.2</td>
<td>5.8</td>
<td>$R_{\text{min}} = 5470$ ft</td>
<td>-</td>
</tr>
<tr>
<td>4000</td>
<td>5.9</td>
<td>$R_{\text{min}} = 4500$ ft</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3500</td>
<td>$R_{\text{min}} = 3710$ ft</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

NC = Normal Crown  
RC = Reverse Crown  
$e_{\text{max}} = 6\%$

### Table 8-7: Superelevation Rates for Horizontal Curves:
Superelevation Rate, $e$ (8%), for Design Speed of
(Metric)

<table>
<thead>
<tr>
<th>Radius (m)</th>
<th>140 km/h</th>
<th>150 km/h</th>
<th>160 km/h</th>
</tr>
</thead>
<tbody>
<tr>
<td>7000</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
</tr>
<tr>
<td>5000</td>
<td>NC</td>
<td>2.1</td>
<td>2.6</td>
</tr>
<tr>
<td>3000</td>
<td>3.0</td>
<td>3.5</td>
<td>4.3</td>
</tr>
<tr>
<td>2500</td>
<td>3.5</td>
<td>4.2</td>
<td>5.1</td>
</tr>
<tr>
<td>2000</td>
<td>4.3</td>
<td>5.2</td>
<td>$R_{\text{min}} = 2015$ m</td>
</tr>
<tr>
<td>1500</td>
<td>5.5</td>
<td>$R_{\text{min}} = 1610$ m</td>
<td>-</td>
</tr>
<tr>
<td>1400</td>
<td>5.7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1300</td>
<td>5.9</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1200</td>
<td>6.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1000</td>
<td>$R_{\text{min}} = 1190$ m</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

NC = Normal Crown  
RC = Reverse Crown  
$e_{\text{max}} = 6\%$

### Table 8-7: Superelevation Rates for Horizontal Curves:
Superelevation Rate, $e$ (8%), for Design Speed of
(US Customary)

<table>
<thead>
<tr>
<th>Radius (ft.)</th>
<th>85 mph</th>
<th>90 mph</th>
<th>95 mph</th>
<th>100 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>23000</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
</tr>
<tr>
<td>20000</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
<td>2.2</td>
</tr>
<tr>
<td>17000</td>
<td>NC</td>
<td>NC</td>
<td>2.2</td>
<td>2.6</td>
</tr>
</tbody>
</table>

NC = Normal Crown  
RC = Reverse Crown  
$e_{\text{max}} = 6\%$
Desirable design values for length of superelevation transition on these facilities are based on using a given maximum relative gradient between profiles of the edge of traveled way and the axis of...
rotation. Table 8-8 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 the cross slope. For superelevation on bridge structures, it is preferred to begin/end superelevation transition at a bridge bent line.

Table 8-8: Maximum Relative Gradient for Superelevation Transition

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Maximum Relative Gradient, %¹</th>
<th>Equivalent Maximum Relative Slope</th>
<th>Design Speed (km/h)</th>
<th>Maximum Relative Gradient, %¹</th>
<th>Equivalent Maximum Relative Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>0.33</td>
<td>1:303</td>
<td>140</td>
<td>0.32</td>
<td>1:313</td>
</tr>
<tr>
<td>90</td>
<td>0.30</td>
<td>1:333</td>
<td>150</td>
<td>0.28</td>
<td>1:357</td>
</tr>
<tr>
<td>95</td>
<td>0.28</td>
<td>1:357</td>
<td>160</td>
<td>0.25</td>
<td>1:400</td>
</tr>
<tr>
<td>100</td>
<td>0.25</td>
<td>1:400</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

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

Vertical curves create a gradual transition between different grades which is essential for the safe and efficient operation of a roadway. The lengths of both crest and sag vertical curves are controlled by the available sight distance.

K Values are calculated using the same equations as in Chapter 3, Section 4.

Design Ks for both crest and sag vertical curves are shown on Table 8-9.

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Stopping Sight Distance (ft)</th>
<th>Crest Vertical Curves</th>
<th>Sag Vertical Curves</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>-</td>
<td>Design K</td>
<td>Design K</td>
</tr>
<tr>
<td>85</td>
<td>1010</td>
<td>473</td>
<td>260</td>
</tr>
<tr>
<td>90</td>
<td>1110</td>
<td>571</td>
<td>288</td>
</tr>
<tr>
<td>95</td>
<td>1220</td>
<td>690</td>
<td>319</td>
</tr>
<tr>
<td>100</td>
<td>1330</td>
<td>820</td>
<td>350</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Speed (km/h)</th>
<th>Stopping Sight Distance (m)</th>
<th>Crest Vertical Curves</th>
<th>Sag Vertical Curves</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>-</td>
<td>Design K</td>
<td>Design K</td>
</tr>
<tr>
<td>140</td>
<td>325</td>
<td>161</td>
<td>84</td>
</tr>
<tr>
<td>150</td>
<td>365</td>
<td>203</td>
<td>96</td>
</tr>
<tr>
<td>160</td>
<td>405</td>
<td>250</td>
<td>107</td>
</tr>
</tbody>
</table>

The length of a sag vertical curve that satisfies the driver comfort criteria is 60 percent of the sag vertical curve lengths required by the sight distance control. The use of driver comfort control should be reserved for special use and where continuous lighting systems are in place.
Section 3 — Roadside Design Criteria

Clear Zone

The clear zone distances are shown in Table 8-10.

Table 8-10: Clear Zone Distances

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Clear Zone Distance (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>80</td>
</tr>
<tr>
<td>90</td>
<td>80</td>
</tr>
<tr>
<td>95</td>
<td>90</td>
</tr>
<tr>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Speed (km/h)</th>
<th>Clear Zone Distance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>140</td>
<td>24</td>
</tr>
<tr>
<td>150</td>
<td>28</td>
</tr>
<tr>
<td>160</td>
<td>30</td>
</tr>
</tbody>
</table>

Slopes

For safety reasons, it is desirable to design relatively flat areas adjacent to the travelway so that out-of-control vehicles are more likely to recover or make a controlled deceleration. Design guide values for the selection of earth fill slope rates in relation to height of fill are shown in Table 8-11. Particularly difficult terrain may require deviation from these general guide values. Where conditions are favorable, it is desirable to use flatter slopes to enhance roadside safety.

Table 8-11 Earth Fill Slope Rates

<table>
<thead>
<tr>
<th>Height of Fill</th>
<th>Usual Max Slope Rate, Vertical:Horizontal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type of Terrain</td>
</tr>
<tr>
<td>Flat or Gently Rolling</td>
<td>Rolling</td>
</tr>
<tr>
<td>0 - 5 ft [0 - 1.5 m]</td>
<td>1V:8H</td>
</tr>
<tr>
<td>5 ft and over [1.5 m and over]</td>
<td>1V:6H</td>
</tr>
</tbody>
</table>

¹ Deviation permitted for particularly difficult terrain conditions
The slope adjacent to the shoulder is called the front slope. Ideally, the front slope should be 1V:8H or flatter, although steeper slopes are acceptable in some locations.

The back slope should typically be 1V:6H or flatter. However, the slope ratio of the back slope may vary depending upon the geologic formation encountered. For example, where the roadway alignment traverses through a rock formation area, back slopes are typically much steeper.

The intersections of slope planes in the highway cross section should be well rounded for added safety and increased stability of out-of-control vehicles. Where barrier is placed on side slopes, the area between the roadway and barrier should be sloped at 1V:10H or flatter.

**Medians**

The median width is the distance between the inside edge of travel lanes of opposing traffic. Median barriers should be considered when the median widths are less than those shown in Table 8-10.
Section 4 — Ramps and Direct Connections

Overview

Ramps and direct connections are designed to the same criteria.

This subsection discusses ramps and direct connections and includes information on the following topics:

- Design Speed
- Lane and Shoulder Widths
- Acceleration and Deceleration Lengths
- Distance Between Successive Ramps
- Grades and Profiles
- Cross Section and Cross Slopes
Design Speed

Similar to facilities with design speeds of 80 mph [130 km/h] or less, ramps on these facilities should also have a relationship between the ramp design speed and the mainlane design speed. The current relationship, in general, is for the ramp design speed to be 85 or 70 percent of the highway design speed, rounded up to the nearest 5 mph [10 km/h] increment, and limiting the speed differential to 10 mph [20 km/h] on the upper range and 20 mph [30 km/h] for the mid-range.

Table 8-12 shows the values for ramp/connector design speeds.

Table 8-12: Guide Values for Ramp/Connection Design Speed as Related to Highway Design Speed¹

<table>
<thead>
<tr>
<th>Ramp Design Speed (mph):</th>
<th>Highway Design Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>85</td>
</tr>
<tr>
<td>Upper Range (85%)</td>
<td>75</td>
</tr>
<tr>
<td>Mid-Range (70%)</td>
<td>65</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ramp Design Speed (km/h)</th>
<th>Highway Design Speed (km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>140</td>
</tr>
<tr>
<td>Upper Range (85%)</td>
<td>120</td>
</tr>
<tr>
<td>Mid-Range (70%)</td>
<td>110</td>
</tr>
</tbody>
</table>

¹ Values determined by calculating the 85 or 70% value of the highway design speed and rounding up to the nearest 5 mph [10 km/h] increment and then adjusting if the rounded value is more than the cap amount from the highway design speed (10 mph [20 km/h] for upper range and 20 mph [30 km/h] for mid range).
Lane and Shoulder Widths

Ramp and Direct Connection shoulder widths (inside and outside) and lane widths are shown in Table 8-13.

Table 8-13: Ramp and Direct Connection Widths

<table>
<thead>
<tr>
<th></th>
<th>(US Customary)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inside Shoulder Width (ft)</td>
<td>Outside Shoulder Width(^1) (ft)</td>
<td>Traffic Lanes (ft)</td>
</tr>
<tr>
<td>1-lane</td>
<td>8</td>
<td>10</td>
<td>14</td>
</tr>
<tr>
<td>2-lane</td>
<td>4</td>
<td>10</td>
<td>26</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>(Metric)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inside Shoulder Width (m)</td>
<td>Outside Shoulder Width(^1) (m)</td>
<td>Traffic Lanes (m)</td>
</tr>
<tr>
<td>1-lane</td>
<td>2.4</td>
<td>3.0</td>
<td>4.3</td>
</tr>
<tr>
<td>2-lane</td>
<td>1.2</td>
<td>3.0</td>
<td>7.9</td>
</tr>
</tbody>
</table>

\(^1\)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 [3.0 m] and the shoulder width on the outside of the curve decreased to 8 ft [2.4 m] (one lane) or 4 ft [1.2 m] (two lane).
Acceleration and Deceleration Lengths

Table 8-14 provides design criteria for exit ramp deceleration and taper lengths. Adjustment factors for grade effects are independent of highway design speed, therefore use Table 3-14 for deceleration length adjustment factors.

Table 8-15 provides design criteria for entrance ramp acceleration and taper lengths; adjustment factors for grade effects are shown in Table 8-16.

<table>
<thead>
<tr>
<th>Highway Design Speed (mph)</th>
<th>Minimum Length of Taper, T (ft)</th>
<th>Deceleration Length, D (ft) for Exit Curve Design Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stop  15  20  25  30  35  40  45  50  55  60  65  70  75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Assumed Exit Curve Speed (mph)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0   14  18  22  26  30  36  40  44  48  52  55  58  61</td>
</tr>
<tr>
<td>30</td>
<td>Existing Criteria in Roadway Design Manual Figure 3-36</td>
<td>--  --  --  --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>35</td>
<td></td>
<td>--  --  --  --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td>--  --  --  --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>45</td>
<td></td>
<td>--  --  --  --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>50</td>
<td></td>
<td>--  --  --  --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>55</td>
<td></td>
<td>--  --  --  --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td>185 --  --  --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>65</td>
<td></td>
<td>225 185 --  --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>70</td>
<td></td>
<td>270 225 190 --  --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>75</td>
<td></td>
<td>310 265 235 195 --  --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>80</td>
<td></td>
<td>335 310 275 240 200 --  --  --  --  --  --  --  --  --</td>
</tr>
<tr>
<td>85</td>
<td></td>
<td>345 360 650 675 660 645 625 600 555 525 490 400 355 325</td>
</tr>
<tr>
<td>90</td>
<td></td>
<td>370 380 780 760 745 725 705 680 640 605 570 450 405 370</td>
</tr>
<tr>
<td>95</td>
<td></td>
<td>425 900 880 865 850 830 805 800 760 730 695 530 485 455</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td>475 490 550 510 575 550 530 510 475 450 425 380 345 315</td>
</tr>
</tbody>
</table>

NOTE: Where providing desirable deceleration length is impractical, it is acceptable to allow for a moderate amount of deceleration (10 mph) within the through lanes and to consider the taper as part of the deceleration length.
Table 8-14: Lengths of Exit Ramp Speed Change Lanes (Metric)

<table>
<thead>
<tr>
<th>Highway Design Speed (km/h)</th>
<th>Minimum Length of Taper, T (m)</th>
<th>Deceleration Length, D (m) for Exit Curve Design Speed (km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stop 20 30 40 50 60 70 80 90 100 110 120 Assumed Exit Curve Speed (km/h)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0 20 28 35 42 51 63 70 77 85 91 98</td>
</tr>
<tr>
<td>50</td>
<td><strong>Existing Criteria in</strong> Roadway Design Manual Figure 3-36</td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>70</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>80</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>90</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td>56 -- -- --</td>
</tr>
<tr>
<td>110</td>
<td></td>
<td>78 52 -- --</td>
</tr>
<tr>
<td>120</td>
<td></td>
<td>102 78 58 --</td>
</tr>
<tr>
<td>130</td>
<td></td>
<td>116 92 73 --</td>
</tr>
<tr>
<td>140</td>
<td>110</td>
<td>144 121 103 80</td>
</tr>
<tr>
<td>150</td>
<td>115</td>
<td>172 150 132 110</td>
</tr>
<tr>
<td>160</td>
<td>130</td>
<td>216 196 180 159</td>
</tr>
</tbody>
</table>

NOTE: Where providing desirable deceleration length is impractical, it is acceptable to allow for a moderate amount of deceleration (15 km/h) within the through lanes and to consider the taper as part of the deceleration length.
Table 8-15: Lengths of Entrance Ramp Speed Change Lanes
(US Customary)

<table>
<thead>
<tr>
<th>Highway Design Speed (mph)</th>
<th>Minimum Length of Taper, T (ft)</th>
<th>Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stop  15 20 25 30 35 40 45 50 55 60 65 70 75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Initial Speed (mph)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0    14 18 22 26 30 36 40 44 48 52 55 58</td>
</tr>
<tr>
<td>30</td>
<td>Existing Criteria in</td>
<td>--   -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>35</td>
<td>Roadway Design Manual Figure 3-36</td>
<td>--   -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td>--   -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>45</td>
<td></td>
<td>--   -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>50</td>
<td></td>
<td>--   -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>55</td>
<td></td>
<td>--   -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td>--   -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>65</td>
<td></td>
<td>132 -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>70</td>
<td></td>
<td>331 70 -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>75</td>
<td></td>
<td>545 287 74 -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>80</td>
<td></td>
<td>771 516 306 79 -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- -- --</td>
</tr>
<tr>
<td>85</td>
<td></td>
<td>345 2186 2154 2045 2006 1945 1828 1601 1426 1227 1009 757 550 326 84</td>
</tr>
<tr>
<td>90</td>
<td></td>
<td>2403 2379 2266 2233 2179 2065 1840 1668 1472 1259 1010 805 584 345</td>
</tr>
<tr>
<td>95</td>
<td></td>
<td>370 2786 2777 2658 2636 2593 2484 2264 2097 1906 1701 1459 1258 1042 808</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td>425 3372 3385 3256 3250 3225 3123 2910 2751 2568 2375 2142 1949 1740 1514</td>
</tr>
</tbody>
</table>

NOTE: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.
### Table 8-15: Lengths of Entrance Ramp Speed Change Lanes (Metric)

<table>
<thead>
<tr>
<th>Highway Design Speed (km/h)</th>
<th>Minimum Length of Taper, T (m)</th>
<th>Acceleration Length, A (m) for Entrance Curve Design Speed (km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stop 20 30 40 50 60 70 80 90 100 110 120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Initial Speed (km/h)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0 20 30 40 47 55 63 70 77 85 91 98</td>
</tr>
<tr>
<td>50</td>
<td>Existing Criteria in Roadway Design Manual Figure 3-36</td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>70</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>80</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>90</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td>-- -- -- --</td>
</tr>
<tr>
<td>110</td>
<td></td>
<td>48  48  48</td>
</tr>
<tr>
<td>120</td>
<td></td>
<td>156 46  46</td>
</tr>
<tr>
<td>130</td>
<td></td>
<td>218 109  --</td>
</tr>
<tr>
<td>140</td>
<td>110 110 703 687 693 652 624 572 507 438 350 245 155 37</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>115 819 806 945 776 750 700 646 581 492 392 305 190</td>
<td></td>
</tr>
<tr>
<td>160</td>
<td>130 977 987 940 928 877 787 726 657 570 496 397</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 400 m.</td>
<td></td>
</tr>
</tbody>
</table>

### Table 8-16: Speed Change Lane Adjustment Factors as a Function of a Grade (US Customary)

<table>
<thead>
<tr>
<th>Design Speed of Roadway (mph)</th>
<th>Ratio of Length on Grade to Length on Level&lt;sup&gt;1&lt;/sup&gt;</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
<th>All Speeds</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3 to 4% Upgrade</td>
<td>3 to 4% Downgrade</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>85</td>
<td>1.62 1.69 1.75 1.80 1.89 1.99 2.10</td>
<td>0.56</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>1.66 1.73 1.80 1.86 1.96 2.08 2.20</td>
<td>0.55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>95</td>
<td>1.71 1.78 1.85 1.92 2.03 2.17 2.30</td>
<td>0.54</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>1.75 1.83 1.90 1.98 2.10 2.26 2.40</td>
<td>0.52</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5 to 6% Upgrade</td>
<td>5 to 6% Downgrade</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>85</td>
<td>2.39 2.51 2.64 2.94 3.15 3.73 4.28</td>
<td>0.46</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>2.50 2.64 2.77 3.10 3.33 4.00 4.65</td>
<td>0.45</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>95</td>
<td>2.62 2.76 2.91 3.27 3.51 4.26 5.03</td>
<td>0.44</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>2.74 2.89 3.04 3.43 3.69 4.53 5.40</td>
<td>0.42</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Distance between Successive Ramps

The minimum acceptable distance between ramps is dependent upon the merge, diverge, and weaving operations that take place between ramps and the *Highway Capacity Manual* should 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 should also be considered.

Grades and Profiles

Grades and profiles are associated with design speed selected for the ramp. Design criteria for design speeds less than 85 mph [140km/h] can be found in Chapter 2.

Cross Section and Cross Slopes

The cross slope for ramp tangent sections should be similar to the cross slope used on the main lanes 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 used and other drainage considerations.

---

Table 8-16: Speed Change Lane Adjustment Factors as a Function of a Grade  
(US Customary)

<table>
<thead>
<tr>
<th>Design Speed of Roadway (km/h)</th>
<th>Ratio of Length on Grade to Length on Level(^1)</th>
<th>3 to 4% Upgrade</th>
<th>3 to 4% Downgrade</th>
<th>5 to 6% Upgrade</th>
<th>5 to 6% Downgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>70</td>
</tr>
<tr>
<td>140</td>
<td></td>
<td>1.57</td>
<td>1.67</td>
<td>1.81</td>
<td>1.79</td>
</tr>
<tr>
<td>150</td>
<td></td>
<td>1.60</td>
<td>1.70</td>
<td>1.86</td>
<td>1.83</td>
</tr>
<tr>
<td>160</td>
<td></td>
<td>1.63</td>
<td>1.73</td>
<td>1.90</td>
<td>1.86</td>
</tr>
<tr>
<td></td>
<td>5 to 6% Upgrade</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>140</td>
<td></td>
<td>2.55</td>
<td>2.82</td>
<td>3.53</td>
<td>3.92</td>
</tr>
<tr>
<td>150</td>
<td></td>
<td>2.70</td>
<td>3.00</td>
<td>3.81</td>
<td>4.29</td>
</tr>
<tr>
<td>160</td>
<td></td>
<td>2.86</td>
<td>3.18</td>
<td>4.09</td>
<td>4.65</td>
</tr>
</tbody>
</table>

\(^1\) Ratio in this table multiplied by length of acceleration distances gives length of acceleration distance on grade.
Appendix A — Longitudinal Barriers

Contents:

Section 1 — Overview
Section 2 — Barrier Need
Section 3 — Structural Considerations of Guard Fence
Section 4 — Placement of Guard Fence
Section 5 — End Treatment of Guard Fence
Section 6 — Determining Length of Need of Barrier
Section 7 — Example Problems
Section 8 — Median Barrier
Section 9 — Emergency Crossovers
Section 1 — Overview

Introduction

The objectives of this appendix are to make available data and guidelines for the use of roadside and median traffic barriers in a consolidated and understandable form. These guidelines should be supplemented by sound engineering judgment.

The area adjacent to the traveled way plays an important role in the safe operation of a high speed facility. Accident statistics show that a significant portion of accidents on rural roads are the single vehicle, run-off-the-road type. Provision of an obstacle free zone and the effective use of barriers to shield obstacles that cannot otherwise be removed or safety treated are important considerations for enhancing safety performance.

The Appendix also contains the following sections:

Section 2 - Barrier Need

Section 3 - Structural Consideration

Section 4 - Placement of Guard Fence

Section 5 - End Treatment of Guard Fence

Section 6 - Determining Length of Need of Barrier

Section 7 - Example Problems

Section 8 - Median Barrier

Section 9 - Emergency Crossovers
Section 2 — Barrier Need

Overview

Traffic barriers are considered only when the obstacle is less forgiving than striking the barrier itself.

Should a roadside obstacle exist, treatment should be considered in the following priority:

1. Remove obstacle.
2. Redesign the obstacle so it can be safely traversed.
3. Relocate the obstacle to a point where it is less likely to be struck.
4. Reduce impact severity by using an appropriate breakaway device.
5. Shield the obstacle with a longitudinal traffic barrier designed for redirection or use a crash cushion.
6. Delineate the obstacle if the above alternatives are not appropriate.

Types of Barrier

**Rigid:** Common rigid barriers used by TxDOT are constant single slope barriers and F-shape barriers that are cast in place or precast. Concrete barriers placed in situ or are keyed into the roadway pavement are considered rigid barriers. Concrete barriers are placed primarily where little movement of the barrier can be tolerated and typically do not deform when impacted. For concrete barriers that are anchored, between 0 and 1 ft. of deflection is anticipated for a TL-3 (high-speed) pickup truck impact. Where considerable truck traffic exists or is anticipated, a TL-4 rated barrier should be considered. (Minimum height of a TL-4 barrier is 36 inches). Concrete barriers are typically used at narrow medians. The exposed ends of the barrier need to be protected by an attenuator.

**Semi-Rigid:** Semi-Rigid barriers commonly used by TxDOT include metal beam guard fence and both pinned and unanchored precast concrete barriers. Semi-Rigid barriers have an expected deflection of between 18" and 60" 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 an (MGS) system is 31 inches plus or minus 1 inch measured from the road surface to top of w beam rail. Metal beam guard fence is used primarily to shield roadside obstacles, such as slopes, utility poles or trees. Precast concrete barriers can be used for temporary or permanent installations. Precast concrete barrier is most commonly used in work zones to shield personnel from traffic.

**Flexible:** High tension cable barriers are the most commonly used flexible barriers. A cable barrier is sometimes referred to as a wire rope safety barrier. It consists of high tension steel cables
mounted on weak posts with a post foundation and typical anchor terminal. Cable barriers are used as median barriers to reduce median crossover vehicle encroachments.

Additional Guidance for each barrier type is provided in subsequent sections.

Applications

The three basic types of obstacles that are commonly shielded using roadside barriers are as follows:

- Slopes, lateral drop-offs, or terrain features,
- Bridge ends and the areas alongside bridges, and
- Other roadside obstacles that cannot be eliminated, made breakaway or otherwise traversable, or relocated.

Table A-1 shows a summary of roadside features that are commonly shielded with guard fence.

<table>
<thead>
<tr>
<th>Roadside Feature</th>
<th>Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Terrain Features:</strong></td>
<td></td>
</tr>
<tr>
<td>Steep Embankment Slope</td>
<td>cz&lt;sup&gt;a&lt;/sup&gt;, See Figure A-1</td>
</tr>
<tr>
<td>Rough Rock Cut</td>
<td>cz</td>
</tr>
<tr>
<td>Boulders</td>
<td>cz, dia. Exceeds 6 in [150 mm]</td>
</tr>
<tr>
<td>Water Body</td>
<td>cz, width exceeds 2 ft [600 mm], permanent</td>
</tr>
<tr>
<td>Lateral Drop-off</td>
<td>cz &amp; steeper than 1V:1H and depth exceeds 2 ft [600 mm]</td>
</tr>
<tr>
<td>Side Ditches</td>
<td>cz &amp; unsafe cross section&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td><strong>Bridges:</strong></td>
<td></td>
</tr>
<tr>
<td>Parapet Wall/Wingwall/Bridge Rail End</td>
<td>approaching traffic</td>
</tr>
<tr>
<td>Area Alongside Bridges</td>
<td>approaching traffic</td>
</tr>
<tr>
<td><strong>Roadside Obstacles:</strong></td>
<td></td>
</tr>
<tr>
<td>Trees</td>
<td>cz &amp; dia. Exceeds 6 in [150 mm]</td>
</tr>
<tr>
<td>Culvert Headwall</td>
<td>cz &amp; size of opening exceeds 3 ft. [900 mm] (w.o. safety grates only)</td>
</tr>
<tr>
<td>Wood Poles, Posts</td>
<td>cz &amp; cross section/area exceeds 50 in&lt;sup&gt;2&lt;/sup&gt; [32000 mm&lt;sup&gt;2&lt;/sup&gt;]</td>
</tr>
<tr>
<td>Bridge Piers, Abutments at Underpasses</td>
<td>cz</td>
</tr>
<tr>
<td>Retaining Walls</td>
<td>cz &amp; not parallel to travelway</td>
</tr>
</tbody>
</table>

<sup>a</sup> cz - Within clear zone for highway class and traffic volume conditions.

<sup>b</sup> For preferred ditch cross sections, see Side Ditches in Chapter 2
The combination of embankment height and side slope rate may indicate barrier protection consideration as shown in Figure A-1. For low fill heights a more abrupt slope rate is tolerable than at high fill heights. Because steeper than 1V:4H side slopes provide little opportunity for drivers to redirect vehicles at high speeds, in the absence of guard fence, a 10 ft area free of obstructions should be provided by the designer beyond the toe of slope.

![Figure A-1](image-url)  

Figure A-1. (US). Guide for Use of Guard fence for Embankment Heights and Slopes (US Customary)

### Working Width

Working width is the distance between the traffic face of the barrier before the impact and the maximum lateral position of any major part of the system or vehicle after the impact. (see Figure A-2) Working width is related to deflection, but working width takes into account the lateral position of the vehicle. Working width should be considered when placing any longitudinal barrier.
Figure A-2. Working Width
Section 3 — Structural Considerations of Guard Fence

Overview

Post spacing, rail shape and thickness, rail height, splice strength and location, post embed-ment, and rail anchorage are all important factors that influence the structural integrity of guard fence.

Post Spacing, Embedment, and Lateral Support

Typical post spacing is 6 ft-3 in [1905 mm] for guard fence. Where guard fence is to be placed at or near the shoulder edge, it is desirable that the roadway crown be widened, typically 2 ft [600 mm] from the back of the post location as shown in Figure A-3, to provide lateral support for the posts. Locating the roadway crown/side slope hinge point behind the rail also provides a platform that increases vehicular stability in the event of impacts that straddle the end section.

Embedment depth is shown on the standard detail sheet for both timber and steel posts.

![Figure A-3. Crown Widening to Accommodate Guardrail](image)

Rail Element

Guard fence is fabricated in a deep beam shape to provide for bending strength. Nominal thickness of the rail is 10 or 12 gauge. End treatments, wingwalls, retaining walls, etc. provide firm rail anchorage. With full splice connections, the anchored rail has sufficient tensile and flexural strength to contain and redirect vehicles under nominal impact conditions.

To insure satisfactory performance for a range of vehicle sizes, rail should be mounted 25 in [635 mm] high as measured from shoulder surface, gutter pan, or widened crown to the center of the rail at the bolt. The rail element shall be spliced midspan between the posts.

Pavement overlays effectively reduce existing rail height. When rail height varies more than 1 in [25 mm] above and 3 in [75 mm] below the 31 in [787 mm] top of rail standard height, steps should be taken to restore the rail to the standard dimension to reduce the possibility of vehicular vaulting
or under riding the system. For existing 28 in [711 mm] rail systems, the rail height shall not vary by more than 2 in [50 mm] above and 1/4 in [6 mm] below the 28 in [711 mm] top of rail. Existing systems installed with a top rail height less than 27 ¾” 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 [787 mm] height, the railing will also need to be adjusted horizontally and an additional post or 9’-4 ½” rail length will be needed to obtain the mid-span splicing location. Existing bridge transitions may need to be upgraded to current standards or adjusted with a new transition section to obtain the 31 in [787 mm] height. The end treatments may require new materials to adhere to the manufacturer’s specifications, such as the breakaway hole and anglestrut locations.

Blockouts

The guard fence is blocked out from the posts with routed timber or composite blockouts (6 in x 8 in [150 mm x 200 mm]). These blockouts minimize vehicle snagging on the posts and reduce the likelihood of a vehicle vaulting over the barrier by maintaining the rail height during the initial stages of post deflection.

It is acceptable to use double blockouts (up to 16 in. [406 mm]) to increase the post offset to avoid obstacles such as curbs. There is no limit to the number of posts that can have double blockouts installed, except terminals, unless approved by the manufacturer. Under special circumstances, such as avoiding buried obstacles that are not relocated, it is also acceptable to install triple blockouts to obtain up to 24 in [610 mm] of clearance for one post for every 75 feet of guard fence.
Section 4 — Placement of Guard Fence

Overview

The placement of guard fence pertains to the lateral and longitudinal position.

Lateral Placement at Shoulder Edge or Curb Face

Typically the face of rail is placed at the shoulder edge or curb face throughout most of its length as shown in Figure A-4.

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 inches [635 mm] above the gutter pan or roadway surface.

Lateral Placement Away From the Shoulder Edge

In certain instances it is desirable to place guard fence closer to the obstacle rather than at the shoulder edge or curb face as shown in Figure A-5. Placement in this manner can substantially reduce the length of rail required to shield a given obstacle and minimize the probability of impact, but undesirably, encroachment angles may increase. This manner of placement is most applicable to small areas of concern such as point type obstacles, overhead sign bridge supports, bridge piers, etc.
To preclude vaulting or impacting at an undesirable position by errant vehicles; care should be exercised in selecting placement location of guard fence with respect to slope conditions. Guard fence may be placed at any lateral location on a side slope only if the slope rate between the edge of the pavement and the face of the barrier is 1V:10H or flatter.

![Figure A-5. Location of Roadside Guard Fence.](image)

**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’-0” [1219 mm] minimum from the back of the post to fixed object or more as diagrammed in Figure A-6. Where conditions permit, a barrier-to-obstacle distance of 5 ft [1524 mm] or more is desirable.
Figure A-6. Allowance for Deflection of Guard fence
Section 5 — End Treatment of Guard Fence

Overview

Guard fence systems must be anchored at both ends to acceptable end treatments, buried terminals, wingwalls, concrete traffic barriers, etc., so that full tensile strength of the rail may be developed.

Approved end treatments have been developed and are recommended for the upstream end of a guard fence system. These approved end treatments shall be used unless the guard fence terminal is located on the downstream end with respect to adjacent traffic of the guard fence and outside the clear zone for opposing traffic (see Figure A-4). In that case a Downstream Anchor Terminal (DAT) section without offset is acceptable for use.
Section 6 — Determining Length of Need of Barrier

Overview

The shape of the obstacle, its location with respect to travel lanes, the volume of traffic and its corresponding clear zone width are the primary variables influencing length of barrier need. Barrier can be considered rigid, and semi rigid such as single slope and F-shape concrete barriers, and metal beam guard fence.

Variables

After all practical means to free the roadside of obstacles have been exhausted, certain areas may remain which constitute an obstacle to errant vehicles. These areas, as illustrated in Figure A-7, will be referred to as an “area of concern.”

Figure A-7. Areas of Concern

Figure A-7 illustrates the variables of interest in the layout of approach barrier to shield an area of concern. Length of need is equal to the sum of the following variables:

- Length of upstream barrier, $L_u$,
- Length of barrier parallel to the area of concern, $L_p$, and
- The length of downstream barrier, $L_d$.

When discussing length of need as it pertains to metal beam guard fence, $L_u$ is the length of guard fence needed to protect traffic adjacent to proposed guard fence. Upstream refers to the guard fence upstream of traffic adjacent to proposed guard fence. While $L_d$ is the length of guard fence needed to protect the opposing traffic. For roadways serving one-way traffic operations, $L_d = 0$. $L_d$ is greater than zero for two-way operations when the area of concern lies within the clear zone of opposing (northbound in Figure A-8) traffic as measured from the centerline pavement markings.
In certain instances, judgment should be exercised to supplement design chart solutions and provide for public safety. For example, high severity fixed objects (e.g., bridge columns) may justify minimum guard fence treatment where located slightly 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'-0" minimum distance away from the obstacle and provide a 10:1 slope for the placement of the metal beam guard fence. If the bridge class culvert is outside the clear zone, Du equals the clear zone distance. If the bridge class culvert is inside the clear zone distance, Du equals the distance to the outside edge of the bridge class culvert.

**Design Equations**

To determine needed length of guard fence 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 [4.9 m] and length of roadside travel of 200 ft [61 m] 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 [4.9 m] and the present ADT is 750 or less, use Equation A-1 for calculating the guard fence length of need.

### Table A-2: Equations for Upstream and Downstream Length of Need

<table>
<thead>
<tr>
<th>ADT ≤ 750</th>
<th>[ L_u = 200 - \frac{200}{D_u} \times G_u ]</th>
<th>Equation A-1.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[ L_d = 200 - \frac{200}{D_d} \times G_d ]</td>
<td>Equation A-2.</td>
</tr>
<tr>
<td>ADT &gt; 750</td>
<td>[ L_u = 250 - \frac{250}{D_u} \times G_u ]</td>
<td>Equation A-3.</td>
</tr>
<tr>
<td></td>
<td>[ L_d = 250 - \frac{250}{D_d} \times G_d ]</td>
<td>Equation A-4.</td>
</tr>
</tbody>
</table>

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)

For low volume conditions, if the clear zone width 16 ft [4.9 m] is met or exceeded, \( L = 0 \).

For higher volumes, a clear zone width of 30 ft [9 m] and length of roadside travel of 250 ft [76 m] are incorporated into the design equation (for use on roadways when the present ADT volume is more than 750 or the recommended clear zone is greater than 16 ft [4.9 m]):

For high volume conditions, if the clear zone width (30 ft [9 m]) is met or exceeded, \( L = 0 \).
The length of need for guard fence, as illustrated below in equation A-5, is equal to the sum of the required upstream length ($L_u$), the guard fence length parallel to the area of concern $L_p$, and the required downstream length.

$$L_{total} = L_u + L_p + L_d$$

*Equation A-5.*

Where:

$L_{total}$ = Length of guard fence needed

$L_u$ = Guard fence Length Upstream of Area of Concern

$L_p$ = Guard fence Length Parallel to Area of Concern

$L_d$ = Guard fence length Downstream from area of concern

**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 center-line markings for two-way traffic operations),
- Design slope conditions, (i.e. will slopes be 1V:10H or flatter?),
- Placement location (alongside shoulder vs. near object, flared, etc.), and
- Presence of other nearby areas of concern which should be considered simultaneously.

Once this design data has been assembled, the appropriate equation 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, of the access location, 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, cost and maintenance associated with the selected treatment and any accident history at the site. Reduced guard fence length to accommodate access points will not require a design exception or a design waiver.

The Example Problems section provides example problems and solutions using the design equations. The guard fence lengths produced by the equations should be rounded up to an even length of guard fence. In circumstances where site conditions permit, the rounded up length of need should terminate at the end of guard fence; any additional length of need component available from an end attenuator should be considered an additional buffer.
Section 7 — Example Problems

Example Problem 1

**Given:** A rural two-lane collector highway containing 6 ft [1.8 m] wide shoulders and a current ADT of 500 is illustrated in Figure A-8. The area of concern is a 16 ft [4.9 m] design clear zone that includes 1V:2H side slopes on a 10 ft [3 m] high embankment section that is 125 ft [38 m] in length alongside the highway.

**Solution:** From the information above and referring to Figure A-1 it is determined that a “rail is needed.” As shown in Equation A-5, the length of need is $L_{total} = L_u + L_p + L_d$. From the given information, $L_p = 125$ ft [38 m]. Because the ADT is less than 750, Equations A-1 and A-2 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 [4.9 m]) clear zone width and the guard fence offset ($G_u$) is 6 ft [1.8 m]. Substituting in Equation A-1.

(US Customary):
A placement of guard fence alongside the 6 ft [1.8 m]-wide shoulder results in $L_u = 125$ ft [38 m].

Referring to Figure A-9, the length of guard fence needed in the downstream is zero because the offset distance from the edge of the travel lane (centerline marking) to the area of concern is greater than the design clear zone (17 ft [5.1 m] greater than 16 ft [4.9 m]). Therefore, $L_d$ is zero.

The design placement is shown in Figure A-10 including 125 ft [38 m] of guard fence adjacent to the obstacle plus 125 ft [38 m] shielding traffic adjacent to proposed guard fence upstream of the obstacle. These lengths of need do not include end treatments.

---

$L_a = 200 - \frac{200}{16} \times 6 = 125$ ft
Example Problem 2

**Given:** A rural two-lane arterial highway containing a shoulder width of 8 ft [2.4 m] and a current ADT of 3500 is illustrated in Figure A-11. The areas of concern are bridge bents located 5 ft [1.5 m] from the edge of shoulder. The side slopes are 1V:6H.

![Figure A-11. Example 2 Problem Layout Rural High Volume.](image)

**Solution:** Referring to Table A-1: General Applications of Conditions for Roadside Barriers bridge piers within the clear zone (30 ft [9 m] in this case) indicates guard fence placement for the north side of the roadway displayed in Figure A-11. As shown in Equation A-5 the length of need is $L_{\text{total}} = L_u + L_p + L_d$. Therefore, $L_p$ is 34 ft [10.4 m] from the given (see Figure A-11) information. Because the ADT is greater than 750, Equations A-3 and A-4 are used to find $L_u$ and $L_d$ (if necessary), respectively:

(US Customary):

$$L_u = 250 - \frac{250}{15} \times 8 = 116.5 \text{ft}$$

Substituting in the equation, the upstream length ($L_u$) is 116.5 ft [35.5 m] if placement is at the shoulder edge.

The downstream (westbound traffic) length of guard fence is also determined by substituting into Equation A-4:

(US Customary):
$L_d = 250 - \frac{250}{27} \times 20 = 65\text{ft}$

$L_d$ is 65 ft [19.8 m] 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 [6 m] (12 ft [3.6 m] lane plus 8 ft [2.4 m] shoulder) from the edge of the travel lane.

Total length of guard fence, $L_u+L_p+L_d$, thus is 116.5 ft [35.5 m] + 34 ft [10.4 m] + 65 ft [19.7 m] or 215.5 ft [65.7 m]; or, rounded to an even length of guard fence, 225 ft [68.6m].

The solution for the south side of the roadway yields the same results; hence placement should be as shown in Figure A-12.

![Figure A-12. Example 2 Problem Solution Guardfence Layout.](image-url)
Example Problem 3

**Given:** A divided (76 ft [23.2 m] median) highway with 4 ft [1.2 m] left and 10 ft [3.0 m] right shoulder widths is illustrated in Figure A-13. The median slopes are 1V:10H, and the outside side slopes are 1V:6H. The cross sectional design allows for the addition of a future lane on the median side of the present lanes. The areas of concern are overhead sign bridge supports offset 25 ft [7.6 m] left and 18 ft [5.5 m] right from edge of the travel lanes as shown below. The ADT is 10,000.

**Solution:** Crash cushions in lieu of guard fence should be considered, particularly for facilities with higher than 10,000 ADT. For this example problem assume crash cushions are not cost effective.

Because the median is sloped at 1V:10H, as shown in Figure A-13, guard fence may be placed thereon (see Figure A-5). Therefore place the guard fence such that the back of the posts are 5 ft [1.5 m] in front of the median overhead sign bridge support to allow for deflection, i.e., 20 ft [6.0 m] from the edge of the travel lanes (including the 1.5 ft [0.5 m] from the back of the post to the face of the rail).

Referring to Equation A-5, \( L_{\text{total}} = L_u + L_p + L_d \). For one-way traffic operations, \( L_d = 0 \); furthermore, for the overhead sign bridge support \( L_p = 0 \). Equation A-3 is used to find \( L_u \) because ADT is greater than 750:

(US Customary):
Lu = 250 – \frac{250}{25} \times 18.5 = 65\text{ft}

Equation A-6

For the median side, \(L_u = 65\text{ ft} [20\text{ m}]\) (rounded to 75 ft [22.9 m] 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 [1.5 m] in front of the fixed object. In contrast, parallel placement at the shoulder edge would have required over 200 ft [60 m] of guard fence.

For the right side of traffic, guard fence must be placed at the shoulder edge (Reference Figure A-5). Substituting in Equation A-3 to determine \(L_u\):

(US Customary):

\[Lu = 250 - \frac{250}{18} \times 10 = 111\text{ft}\]

Equation A-7

Using parallel placement for the entire length, \(L_u = 111\text{ ft} [34\text{ m}]\) (which should be rounded to 125 ft [38] to conform to even lengths of guard fence).

Using parallel placement for the entire length of guard fence for both the median and left side, placement is as shown in Figure A-14.

![Figure A-14. Example 3 Problem Solution Guard Fence Layout.](image)
Section 8 — Median Barrier

Overview

Median barriers are used to reduce median crossover vehicle encroachments and to protect against continuous longitudinal obstacles, and can generally be categorized as:

- Concrete barriers (such as F-shape or single sloped), or
- High-tension cable barrier systems.

The utilization of other median barriers, such as metal beam guard fence, may be appropriate based on the need to protect point obstacles in the median, such as overhead sign supports, etc. (See Sections 1-7).

Application

On high-speed highways, median barriers should be considered based on the criteria shown in Table 3. Flush medians or frequent crossovers may preclude the use of median barriers based on an engineering analysis of individual 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 edge of pavement to edge of pavement),
  - Traffic volumes, including estimated traffic growth and percent trucks,
  - Types and severity of crashes,
  - Posted speed limit,
Continuous barrier should be limited to areas where it is needed to reduce cross median incidents and should not be used for point obstacles (i.e. overhead sign bridges, etc.), or in areas of lesser concern (i.e. wider medians, forested areas, etc.).

**Placement**

As a general rule, a barrier should be placed as far from the traveled way as possible while maintaining the proper operation and performance of the system. The more lateral offset afforded a driver, the better the opportunity for the driver to regain control of the vehicle in a traversable median and avoid a barrier impact. The placement of concrete barrier adjacent to narrow shoulders is discouraged. It is recommended that a minimum clear distance of 12 ft be maintained from the edge of the travel lane and a cable barrier to prevent incidental contact with the barrier being too close to the travel lane. Also, due to the deflection of the cable barrier, it is recommended that a minimum distance of 12 ft be maintained between the cable barrier and any obstruction being protected.

**Slopes**

Where possible, barriers should be installed on relatively flat, unobstructed terrain (1V:10H or flatter). Barriers may also be placed on 1V:6H maximum slopes as shown in Figure A-15. The centerline axis of the barrier shall be vertical.

From the perspective of barrier performance alone, it is acceptable placement practice to locate the barrier at, or within 1 ft. [0.3m], 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. [2.4 m] from the bottom of the median ditch line. This offset from the bottom of the ditch line reduces the potential for the vehicle to strike 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 regrading or other options are not feasible, placement of cable barrier on slopes up to 1V:4H is an alternative to consider. Designers should contact the Design Division for assistance in locating the
cable barrier at the proper location along the steeper slopes. While not desirable, some median configurations may require barrier placement on both sides of the median to provide the proper protection.

![Desirable Barrier Placement in Non-Level Medians](image)

**Figure A-15. Desirable Barrier Placement in Non-Level Medians.**

### Additional Cable Median Barrier Guidance

The cable barrier is for median use only and on medians greater than 25 ft. Median widths of 25 ft or less require the use of a more rigid barrier such as concrete median barrier.
The selection of Test Level 4 (TL-4) cable barrier over Test Level 3 (TL-3) cable barrier is at the district's option. FHWA policy requires that all roadside appurtenances such as traffic barrier and barrier terminals used on the National Highway System meet the performance criteria contained in the National Cooperative Highway Research Program (NCHRP) Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features or the updated testing procedures found in The Manual for Assessment of Safety Hardware (MASH). Safety features approved to a minimum TL-3 are acceptable for highspeed arterial highway. Any decision to use TL-4 tested barrier should be made based on-site conditions and traffic mix, using engineering judgement.

The vertical alignment of the system is essential since the location of the bottom cable with respect to the ground is critical to capturing smaller vehicles. Special attention should be placed on sag vertical alignments. The cables and/or posts placed in sockets are free standing (not held down by the system) and will come to a taut elevation between two tangent points when the cable is tensioned creating a larger distance from the ground line to the bottom cable than allowed by the manufacturer's installation manual. Sag vertical alignments with radii of less than a K-Value of 11 should be avoided.

The placement of the system should also take into consideration the drainage facilities located in the median. Cross drainage structures with less than 36” of cover pose a challenge for placing posts. Structures of less than 16 feet 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’ clearance from the proposed cable barrier to the obstruction, the MBGF may be removed. If there is less than 12’ clearance from the proposed cable barrier to the obstruction, it is recommended that the MBGF be left in place, and the cable barrier be placed such that there is a minimum of 4.0’ (5’ preferred) from the back of the MBGF posts to the barrier. This allows for deflection of the MBGF without engaging the cable barrier. Cable barrier should be a minimum 5’ behind SGT's to allow for extrusion and gating of the end treatment.

Cable barrier systems deflection is based on it being installed along a tangent or when struck on the “concave” side (from the inside of a curve). When it is struck in the “convex” side (from the outside of the curve) the barrier must deflect enough to redevelop a concave condition. Placement of the barrier on the convex side is also recommended to allow maximum median availability for deflection (see Figure A-16). In order to minimize the length over which this occurs, closer post spacing through these curves is recommended (see Table A-4).
A recommended maximum run of cable barrier between anchors should be approximately 10,000 ft. This length allows for proper tensioning of the system and reasonable construction installation time to get a run in operation. Runs of shorter and longer lengths between anchors may be appropriate in specific locations and each run should be determined to meet the field situations.
Section 9 — Emergency Crossovers

Overview

Emergency crossovers may be provided when needed to facilitate emergency and law enforcement vehicles. Coordination with local and state law enforcement and emergency services personnel is recommended to identify roadway sections where crossovers may be necessary.

Location

When selecting a location for a crossover, the following guidance should be used:

- Do not install emergency crossovers in urban locations. Interchanges are closely spaced and provide opportunities for making needed turn movements.

- Emergency crossovers should be spaced at approximately 2 mile intervals, except where coordination with local and state law enforcement has identified a need for spacing of crossovers of less than 2 mi. [3.2 km] to address local issues.

- Emergency crossovers should be placed at reasonable intervals based on engineering judgment and safety, generally no closer than ½ mi. [0.8 km] between crossovers.

- The emergency crossover is not to be located within 1500 ft. [457.2 m] from any ramp terminal or other access connection.

- The emergency crossover is not to be located within curves requiring superelevation, unless field engineering determines the location is safe and reasonable for emergency use.

- Emergency crossovers should be located where more than minimum stopping sight distance is provided.

Construction

When ending a run of cable barrier, the cable barrier terminals should be located, when possible, behind some protection such as the MBGF, leaving adequate distance to allow an emergency vehicle to maneuver around if necessary. See Figure A-17.
Figure A-17. Cable Terminals Behind Metal Beam Guard Fence

The terminals can be placed in locations with no protection, but since they provide the anchorage for the cable barrier system, protecting them from possible hits is recommended. These terminals are also gating (meaning they will not prevent a vehicle from going through).

When switching the cable barrier from one median side to the other and the terminals are not protected, overlapping the runs of cable barrier is recommended to provide adequate protection from possible crossovers if the median is wide enough to allow emergency vehicles to utilize it as an effective emergency crossover. (See Figures A-18 through A-19). The cable barrier anchor terminal should be protected behind guardrail or other barrier from potential impacts. An exposed cable barrier anchor terminal will render the cable system ineffective if damaged by an impact.

Figure A-18. Recommended Cable Barrier Lap Length
Another typical layout for emergency crossovers may be as shown below.

Figure A-19. Another Typical Layout for Emergency Crossovers

- Emergency crossovers should be an all-weather surface. It is recommended that they be constructed with a surface treatment that does not invite use. Grade 1 or 2 aggregate or bladed recycled asphalt pavement (RAP) has provided an adequate low cost surface in some applications.

- Emergency crossovers should be approximately 20 ft. [6.1 m] with return radii of 10 ft. [3.0 m]. Wider crossovers invite non-emergency use and should only be constructed after an engineering study of the site.

- To be inconspicuous to main lane traffic, the surface should be depressed below the shoulder level, if possible.
Appendix B — Treatment of Pavement Drop-offs in Work Zones

Contents:

Section 1 — Overview
Section 1 — Overview

Scope

These guidelines apply to construction zone work where continuous pavement edges or drop-offs exist parallel and adjacent to a lane used for traffic. These guidelines do not apply to short term operations. The Texas Manual on Uniform Traffic Control Devices (TMUTCD) defines short term operations as daytime work from one to twelve hours.

These guidelines do not constitute a rigid standard or policy; rather, they are guidance to be used in conjunction with engineering judgment.

Types of Treatment

Treatment may consist of either or both of the following:

◆ Warning devices (such as signs or channelizing devices), and
◆ Protective barriers (such as concrete traffic barriers or metal beam guard fence).

Factors Affecting Treatment Choice

The type of treatment (warning device or protective barrier or both) selected depends on several factors, including engineering judgement. These guidelines are based on the following factors:

Factors Considered in the Guidelines

<table>
<thead>
<tr>
<th>Factor</th>
<th>Definition</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Edge condition</td>
<td>Slope of the drop-off</td>
<td>For more information, see “Edge Condition” subheading below.</td>
</tr>
<tr>
<td>Lateral clearance</td>
<td>Distance from the edge of the travel lane to the edge condition</td>
<td>See Figure B-1 for description.</td>
</tr>
<tr>
<td>Edge height</td>
<td>Depth of the drop-off</td>
<td>See Figure B-1 for description.</td>
</tr>
</tbody>
</table>
In addition to the factors considered in the guidelines, each construction zone drop-off situation should be analyzed individually, taking into account other variables, such as:

- Traffic mix,
- Posted speed in the construction zone,
- Horizontal curvature, and
- Practicality of treatment options.

In urban areas where speeds of 30 mph [50 km/h] or less can be predicted for traffic in a particular construction zone, there may be a lesser need for signing, delineation, and barriers. Even so, sharp 90 degree edges greater than 2 inches [50 mm] in height, if located within a lateral offset distance of 6 feet [1.8 m] or less from a traffic lane, may indicate a higher level of treatment.

If distance $Y$ (as described in Figure B-1) must be less than 3 feet [0.9 m], use of positive barrier may not be feasible. In such a case, if a positive barrier is needed (according to Figure B-2), then consider one of the following:

- Moving the lane of travel laterally to provide the needed space, and
- Providing an edge slope such as Edge Condition I.
**Edge Condition**

“Edge condition” refers to the slope of the drop-off. The following table describes three edge condition types used in these guidelines. These edge conditions may be present between shoulders and travel lanes, between adjacent or opposing travel lanes, or at intermediate points across the width of the paved surface. Due to the variability in construction operations, tolerances in the dimensions shown in the figures may be allowed by the engineer.

### Edge Condition Types

<table>
<thead>
<tr>
<th>Condition Type &amp; Description</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Edge Condition I</td>
<td>Most vehicles are able to traverse an edge condition with a slope rate of 3 to 1 (horizontal to vertical) or flatter. The slope must be constructed with a compacted material capable of supporting vehicles.</td>
</tr>
<tr>
<td>$S = 3:1$ or flatter slope rate ($H:V$)</td>
<td></td>
</tr>
<tr>
<td>Edge Condition II</td>
<td>Most vehicles are able to traverse an edge condition with a slope between 2.99 to 1 and 1 to 1 (horizontal to vertical) as long as $D$ does not exceed 5 inches [125 mm]. Undercarriage drag on most automobiles will occur as $D$ exceeds 6 inches [150 mm]. As $D$ exceeds 24 inches [0.6 m], the possibility of rollover is greater for most vehicles.</td>
</tr>
<tr>
<td>$S = 2.99:1$ to 1:1 slope rate ($H:V$)</td>
<td></td>
</tr>
<tr>
<td>Edge Condition III</td>
<td>Slopes steeper than 1 to 1 (horizontal to vertical) where $D$ is greater than 2 inches [50 mm] can present a more difficult control factor for some vehicles, if not properly treated. For example, in the zone where $D$ is greater than two up to 24 inches [50 mm to 0.6 m] different types of vehicles may experience different steering control at different edge heights. Automobiles might experience more steering control differential in the greater than 2 up to 5 inch [50 to 125 mm] zone. Trucks, particularly those with high loads, have more steering control differential in the greater than 5 up to 24 [50 mm to 0.6 m] zone. As $D$ exceeds 24 inches [0.6 m], the possibilities of rollover is greater for most vehicles.</td>
</tr>
<tr>
<td>$S$ is steeper than 1:1 slope rate ($H:V$)</td>
<td>NOTE: Milling or overlay operations that result in Edge Condition III should not be in place without appropriate warning treatments, and these conditions should not be left in place for extended periods of time.</td>
</tr>
</tbody>
</table>

### Guidelines for Treatment

The following guidelines show the recommended treatment for given combinations of edge condition, lateral clearance, and edge height. Remember to consider other factors listed above and use engineering judgment.
### Treatment Guidelines for Pavement Drop-offs in Construction Work Zones

<table>
<thead>
<tr>
<th>Edge Condition</th>
<th>Lateral Clearance</th>
<th>Edge Height</th>
<th>Usual Treatment (See Note 3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>30 ft. [9 m]</td>
<td>0 to 1 in.  [0 to 25 mm]</td>
<td>no treatment</td>
</tr>
<tr>
<td>(slope is 3:1 or flatter)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;1 to 2 in. [&gt;25 to 50 mm]</td>
<td>CW 8-11 signs</td>
</tr>
<tr>
<td>II</td>
<td>20 ft. [6 m]</td>
<td>0 to 1 in.  [0 to 25 mm]</td>
<td>no treatment</td>
</tr>
<tr>
<td>(slope is between 2.99:1 and 1:1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;1 to 2 in. [&gt;25 to 50 mm]</td>
<td>CW 8-11 signs</td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;2 to 5 in. [&gt;50 to 125 mm]</td>
<td>CW 8-9a or CW 8-11 signs plus channelizing devices</td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;5 to 24 in. [&gt;125 to 600 mm]</td>
<td>CW 8-9a or CW 8-11 signs plus drums (see Note 1)</td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;24 in. [&gt;600 mm]</td>
<td>Check indications for positive barrier (See Note 2)</td>
</tr>
<tr>
<td>-</td>
<td>&gt;20 ft. but 30 ft. [&gt;6 m but 9 m]</td>
<td>0 to 1 in. [0 to 25 mm]</td>
<td>no treatment</td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;1 to 2 in. [&gt;25 to 50 mm]</td>
<td>CW 8-11 signs</td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;2 in. [&gt;50mm]</td>
<td>CW 8-9a or CW 8-11 signs plus channelizing devices</td>
</tr>
<tr>
<td>-</td>
<td>&gt;30 ft. [&gt;9 m]</td>
<td>Any height</td>
<td>no treatment</td>
</tr>
<tr>
<td>III</td>
<td>20 ft. [6 m]</td>
<td>0 to 1 in.  [0 to 25 mm]</td>
<td>no treatment</td>
</tr>
<tr>
<td>(slope is steeper than 1:1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;1 to 2 in. [&gt;25 to 50 mm]</td>
<td>CW 8-11 signs</td>
</tr>
<tr>
<td>-</td>
<td></td>
<td>&gt;2 to 24 in. [&gt;50 to 600 mm]</td>
<td>CW 8-9a or CW 8-11 signs plus drums (see Note 1)</td>
</tr>
</tbody>
</table>
Use of Positive Barriers

The use of the pavement drop-offs treatment guidelines provides a practical approach to the use of positive barriers for the protection of vehicles from pavement drop-offs. Other factors, such as the presence of heavy machinery, construction workers, or the mix and volume of traffic, may make positive barriers appropriate, even when the edge condition alone may not justify the barrier.

NOTE: An approved end treatment should be provided for any positive barrier end located within a lateral offset of 20 feet [6.0 m] from the edge of the travel lane.
Figure B-2. Conditions Indicating Use of Positive Barrier.
Appendix C — Driveway Design Guidelines

Contents:

Section 1 — Purpose
Section 2 — Introduction
Section 3 — Driveway Design Principles
Section 4 — Profiles
Section 5 — Driveway Angle
Section 6 — Pedestrian Considerations
Section 7 — Visibility
Section 8 — References
Section 1 — Purpose

The purpose of this Appendix is to provide guidance on the location and design of driveway connections.

Because field conditions are highly variable with respect to driveways, the guidance provided herein may not always be completely applicable. Therefore, departures from this design guidance for driveways to meet field conditions are expected and do not require or constitute a need for any type of design exception or design waiver; however, they may require support documentation in the form of traffic operations and safety analysis.

Additional information can also be found in the Access Management Manual for permitting guidelines and for additional access discussion.
Section 2 — Introduction

General Guidelines

Driveways provide the physical transition between the public highway and the abutting property. Driveways should be located and designed to minimize negative impacts on traffic operations while providing safe entry and exit to/from the abutting property. The location and design of the driveway should take into account characteristics of the roadway, the abutting property and the potential users. In order to assure that driveways provide for safe and efficient traffic movements, it is necessary to consider the driveway's critical dimensions and design features. This Appendix applies to new driveways, and modification of existing driveways.

Definitions

1. **Apron**: On curb and gutter sections, that part of a driveway from the pavement to a selected point that is usually 6 inches in elevation above the edge of pavement (although it may vary by location or roadway) or to the right-of-way, which ever is greater. On sections with a drainage ditch, that part of a driveway from the edge of pavement to the right-of-way line.

2. **Commercial Driveway**: A driveway for use by trucks (typically SU or larger design vehicles as defined by AASHTO) to deliver merchandise to a retail outlet and/or for use by service vehicles, such as for solid waste collection.

3. **Divided Driveway**: A driveway providing a raised or depressed median, between the ingress/egress sides of a driveway. Medians can be painted (fully traversable) when curbing is not allowed within the right-of-way, slightly raised curb (mountable) when U-turns are allowed or curbed (traversable) when U-turns are not allowed.

4. **Driveway**: A driveway is an access constructed within a public right-of-way, connecting a public roadway with adjacent property and intended to provide vehicular access. The applicable section of this chapter governs the geometric design criteria for driveways.

5. **Effective Turning Radius**: The minimum radius appropriate for turning from the right-hand travel lane on the approach street to the appropriate lane of the receiving street. This radius is determined by the selection of a design vehicle appropriate for the streets being designed and the lane on the receiving street into which that design vehicle will turn. Desirably this should be at least 25 ft [7.5 m].

6. **Farm/Ranch Driveway**: A 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.
7. **Field Driveway**: A limited-use driveway for the occasional/infrequent use by equipment used for the purpose of cultivating, planting and harvesting or maintenance of agricultural land, or by equipment used for ancillary mineral production.

8. **Non-Residential/Commercial Driveway**: Driveway having a traffic volume in excess of 20 vehicles per day and is not a Public Street/Road or a Residential Driveway.

9. **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.

10. **One-way Driveway**: A driveway designed for either an ingress/egress maneuver but not both.

11. **Public Driveway (Streets and Roads)**: A driveway providing ingress/egress from a roadway for which the right-of-way is deeded to and the roadway maintenance is performed by a village, town, city, county or municipal utility district.

12. **Radial Return or Flare Drop Curb**: For Residential Driveways onto Collector and Local streets is maximum of 10 feet and minimum of 3 feet. A radial return is always used where the posted or operating speed is greater than 45 mph and the design vehicle type exceeds 30 feet in length.

13. **Residential Driveway**: A driveway serving a single-family residence or duplex and has less than 20 vehicles per day using the driveway.

14. **Service Driveway**: A driveway for occasional or infrequent use by vehicles or equipment to service an oil or gas well, electric substation, water well, water treatment plant, sewage lift station, waste water treatment plant, detention basin, water reservoir, emergency services, automated or remotely controlled pumping station, logging road, and other activities that may be identified by TxDOT.

15. **Shared Driveway**: A driveway shared by adjacent owners.

16. **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.

17. **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.

18. **Throat Width**: The driveway width measured at the end of the return radii. Refer to Figure C-2.
Section 3 — Driveway Design Principles

General Guidelines

The following guidelines apply to all driveways to a state highway.

1. The driveway placement should be such that drivers approaching from the main roadway will have sufficient sight distance to ascertain the driveway’s location in order to safely decelerate and complete the entry maneuver. Also, the driveway placement should be such that an exiting driver will have sufficient sight distance to judge a safe gap in oncoming traffic. For selecting appropriate driveway spacing distance, refer to the TxDOT Access Management Manual.

2. Each driveway radius should accommodate the appropriate design vehicle. This will generally be the passenger car (AASHTO P design vehicle) unless the driveway will routinely be expected to handle more than four larger vehicles per hour. Examples of facilities for which a larger design vehicle would normally be appropriate include truck terminals, bus terminals, and connections that serve the loading docks of shopping centers. Figure C-1 illustrates the effects of the radius on the right-turn entry and exit maneuver.

3. Figure C-2 illustrates the driveway design elements including return radius, entry width, exit width, throat width, and throat length.

4. 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.

5. Driveways where pedestrian traffic is a potential, shall be designed to maintain an accessible pedestrian route that is at least four feet wide across the driveway (see Section 6).

6. One-way driveways should have a minimum throat length of 50 feet (15 m) and preferably 75 feet (23 m).

Figure C-1. Effects of Return Radius on the Right-Turn Maneuver
Appendix C — Driveway Design Guidelines

Geometrics for Two-Way Driveways

The following are standards for two-way driveways.

1. Private Residential Driveway – Driveways serving single-family or duplex residences are normally designed as non-simultaneous two-way driveways. Standard design criteria for private residential driveways are provided in Table C-1. However, for existing cases where the criteria cannot be obtained, every attempt should be made to match the existing driveway width at the ROW line.

2. Commercial Driveways – At locations where the expected volume of large vehicles is four or more per hour, the design should be based on the appropriate design vehicle. Such situations 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.

<table>
<thead>
<tr>
<th>Radius (ft.)</th>
<th>Throat Width</th>
<th>Metric Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Standard (ft.)</td>
<td>Maximum (ft.)</td>
</tr>
<tr>
<td>15</td>
<td>14</td>
<td>24</td>
</tr>
</tbody>
</table>

Figure C-2. Driveway Design Elements
Two exit lanes are recommended when the expected driveway exit volume exceeds 200 vph.

In cases where one-way operation is appropriate, a condition of the driveway permit should require that appropriate one-way signing be installed and maintained.

Table C-2 provides standard design criteria for two-way commercial driveways that would be expected to accommodate only P and SU design vehicles. Driveway Designs for larger vehicles will be considered on a case by case basis.

### Table C-2. Designs for Two-Way Commercial Driveways

<table>
<thead>
<tr>
<th>Condition</th>
<th>US Customary Units</th>
<th>Metric Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Radius (R) (ft)</td>
<td>Radius (R) (m)</td>
</tr>
<tr>
<td></td>
<td>Throat Width (W) (ft)</td>
<td>Throat Width (W) (m)</td>
</tr>
<tr>
<td>One entry lane and one exit lane, fewer than 4 large vehicles per hour</td>
<td>25</td>
<td>7.5</td>
</tr>
<tr>
<td>(see Fig. C-3)</td>
<td>28</td>
<td>8.4</td>
</tr>
<tr>
<td>One entry lane and one exit lane, 4 or more SU vehicles per day</td>
<td>30</td>
<td>9.0</td>
</tr>
<tr>
<td>(see Fig. C-3)</td>
<td>30</td>
<td>9.0</td>
</tr>
<tr>
<td>One entry lane and two exit lanes, without divider (see Fig. C-4)</td>
<td>25</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>12.0</td>
</tr>
<tr>
<td>One entry lane and two exit lanes, with divider (see Fig. C-5)</td>
<td>25</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td>44(1)-55(2)</td>
<td>13.2(1)-15.0(2)</td>
</tr>
<tr>
<td>Two entry lanes and two exit lanes, with divider (see Fig. C-6)</td>
<td>25</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td>56(1)-67(2)</td>
<td>16.8(1)-18.9(2)</td>
</tr>
</tbody>
</table>

(1) See Table C-3 for minimum divider widths
(2) See Table C-3 for maximum divider widths

3. 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.

4. 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.

5. Farm/Ranch Driveway – A typical design for a farm/ranch driveway should provide a 25-foot return radii and a 20-foot throat width. The distance from the edge of pavement must be sufficient to store the longest vehicle, or combination of vehicles, expected. At a minimum, this will normally be a truck with trailer.
Figure C-3. One Entry Lane/One Exit Lane

Figure C-4. One Entry Lane/Two Exit Lanes (Without a Divider)

Figure C-5. One Entry Lane/Two Exit Lanes (With a Divider)

Figure C-6. Two Entry Lanes/Two Exit Lanes (With a Divider)
Divided Driveways

A raised or depressed separation between the entry and exit sides of a divided driveway needs to be visible to drivers. Suggested treatments and divider sizes are shown in Table C-3:

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Width</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slightly raised (4in [100 mm]) with contrasting surface</td>
<td>4 ft (1.2 – 4.5 m)</td>
<td>20 ft [6.0 m]</td>
</tr>
</tbody>
</table>

(1) For Rural - Rounded edges, 30° to 45° slope. (See Figure C-7)

(2) 6 ft. for Ped. Refuge

Figure C-7 illustrates a slightly raised divider (height 4 inches).

Figure C-7. Illustration of Slightly Raised Divider

A divided driveway is desirable in the following situations:

1. There are a total of four or more entering and exiting lanes.
2. A large number of pedestrians (30 or more in a one-hour interval) routinely cross the driveway.

Locating signing 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 feet [4.5 m]. On the other hand, a divider that is too small may not be adequately visible to the motorist. Therefore the recommended minimum width of a slightly raised divider (height > 4 inches) is 4 feet [1.2 m], and 6 feet for a Ped. Refuge.
Section 4 — Profiles

Public driveways and commercial driveways should be constructed with a vertical curve between the pavement cross-slope and the driveway approach and between changes in grade within the driveway throat length. A private residential driveway may be constructed without vertical curves provided that a change in grade does not adversely affect vehicle operations. Typically a change in grade of three percent (3%) or less and a distance between changes in grade of at least eleven feet [3.3 m] accommodates most vehicles. However, literature suggests that a six percent (6%) to eight percent (8%) change in grade may operate effectively. Individual site conditions should be evaluated to accommodate the vehicle fleet using the driveway.

Driveway Grades

To achieve satisfactory driveway profiles, some of the significant factors to be considered are:

1. Abrupt grade changes, which cause vehicles entering and exiting driveways to move at extremely slow speeds, can create:
   - The possibility of rear end collisions for vehicles entering the driveway, and
   - The need for large traffic gaps that may be unavailable or infrequent, causing drivers to accept inadequate gaps.

2. Where sidewalks are present, or in developing areas where pedestrians may be expected now or in the future, slower turning speeds may be beneficial and special design requirements apply. See Section 6 for more information.

3. The comfort of vehicle occupants and potential vehicle damage, (i.e. prevent the dragging of center or overhanging portion of passenger vehicles).

4. Grades must be compatible with the site requirements for sight distance and drainage, to prevent excessive drainage runoff from entering the roadway or adjacent property.

Because a large combination of slopes, tangent lengths, and vertical curves will provide satisfactory driveway profiles, some generalizations should be considered relative.

On curb and gutter sections, placement of vertical curves should be at the extended gutter line and not closer to the travel lanes unless curb and gutter returns and proper drainage are provided. On curb and gutter sections, 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.
As shown in Table C-4, the suggested changes in driveway grades with a vertical curve (between the pavement cross slope and the driveway apron slope) are approximately 10 percent for private residential driveways and approximately 8 percent for all other driveways.

Table C-4. Suggested Change in Grade with a Vertical Curve

<table>
<thead>
<tr>
<th>Driveway</th>
<th>Change in Grade (A)$^{(1)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Private Residential Driveways</td>
<td>10%</td>
</tr>
<tr>
<td>All Other Driveways</td>
<td>8%</td>
</tr>
</tbody>
</table>

$^{(1)}$ Change in grade between the pavement cross-slope and the driveway apron slope

Construction practice can provide a suitable sag vertical curve between the pavement cross-slope and the driveway apron when the apron length $L_a$ (see Figure C-8) is equal to or greater than 20 feet [7 m].

Figure C-8. Suggested Dimensions to Achieve an Appropriate Vertical Curve

Maximum driveway grades should be limited to 12 percent for private residential driveways and to 8 percent for other driveways. Where possible, the driveway grade should be limited to 6 percent or less within the roadway right-of-way.

A construction easement is required for construction beyond the right-of-way line. For construction beyond the right-of-way, it is necessary for the property owner to furnish the construction easement or right of entry required.

Also, within the limits of curb return radii, no drop curb should be allowed except as required for curb ramps.
The length of the vertical curve between the pavement cross-slope and the driveway apron is a function of the algebraic difference in the grades. Table C-5 provides the desirable and minimum lengths for these vertical curves.

**Table C-5. Length of Vertical Curve L (feet) For a Change in Grade Between the Pavement Cross-Slope and the Driveway Apron Slope**

<table>
<thead>
<tr>
<th>Change in Grade, A</th>
<th>Crests</th>
<th>Sags</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Des. ft (m)</td>
<td>Min. ft (m)</td>
</tr>
<tr>
<td>4-5%</td>
<td>5 (1.5)</td>
<td>3 (0.9)</td>
</tr>
<tr>
<td>6-7%</td>
<td>6 (1.8)</td>
<td>4 (1.2)</td>
</tr>
<tr>
<td>8-10%</td>
<td>8 (2.4)</td>
<td>5 (1.5)</td>
</tr>
</tbody>
</table>

Rounded: Parabolic curvature. The plans may specify a particular type of curvature.
Des.: Desirable Minimum Length
Min.: Minimum Length
Where practical, greater lengths should be provided to achieve a flatter and smoother profile.

C-9 through C-11 illustrate typical driveway profiles.

The length of the vertical curve at other points of driveway grade change is also a function of the algebraic difference in the grades. Table C-6 provides the typical lengths for these vertical curves.

Figures C-9 through C-11 illustrate typical driveway profiles.

**Table C-6. Typical Length of Vertical Curve, L, For Change in Grade in Driveway Profile**

<table>
<thead>
<tr>
<th>Change in Grade, A</th>
<th>Crest</th>
<th>Sag</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Private Residential Driveways</td>
<td>Other Driveways</td>
</tr>
<tr>
<td></td>
<td>ft (m)</td>
<td>ft (m)</td>
</tr>
<tr>
<td>4-5%</td>
<td>2 (0.6)</td>
<td>5 (1.5)</td>
</tr>
<tr>
<td>6-7%</td>
<td>3 (0.9)</td>
<td>5 (1.5)</td>
</tr>
<tr>
<td>8-10%</td>
<td>4 (1.2)</td>
<td>6 (1.8)</td>
</tr>
</tbody>
</table>
Profiles on Curb and Gutter Sections

Figure C-9. Roadway with Curb and Gutter, Driveway Profiles on an Upgrade

Figure C-10. Roadway with Curb and Gutter, Driveway Profiles on a Downgrade

See Tables C-5 and C-6 for lengths of vertical curves.

Profiles with Drainage Ditch

Figure C-11. Driveway Profiles on Roadway with Drainage Ditch

See Tables C-5 and C-6 for lengths of vertical curves.
Section 5 — Driveway Angle

Two-way driveways 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. Suggested limiting values on driveway angles are:

**Residential Driveway:** 75°

**Commercial Driveway:** 75°; commercial driveways expected to have a volume of 400 vehicles per day or two or more trucks/large vehicles in a one-hour period shall be designed as normal intersections (public driveway).

**Normal Intersection (Public Driveway), Service Driveway and Field Driveway:** 80°.

The angle of intersection between the centerline of a one-way driveway and the edge of pavement of the public roadway may be between forty-five (45°) and ninety degrees (90°). Sixty degrees (60°) is a commonly used angle for one-way driveways.
Section 6 — Pedestrian Considerations

General Guidelines

Accommodating pedestrians and vehicular traffic at the junctions of sidewalks and driveways presents a variety of challenges. Some general principles are:

◆ The maximum cross-slope at any point on a sidewalk (including the crossing of a driveway) is two percent (2%).

◆ 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.

◆ Consider using a triangular island for pedestrian refuge in a high-volume driveway. The minimum refuge area is 5 feet x 6 feet and preferably larger. (See Figure C-12).

◆ Locate sidewalks far enough from the curb, or edge of pavement, to provide a suitable vertical curve transition between the pavement cross-slope and the driveway apron and to allow the driveway to cross the sidewalk at the sidewalk’s normal elevation (see Section 4, Profiles on Curb and Gutter Sections for illustrations of driveway profiles.)

◆ Where driveways are closely spaced, consider the use of right-in/right-out driveways to eliminate conflicts between left-turning vehicles and pedestrians and bicyclists. 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.

◆ Provide adequate throat length so that a vehicle backing out of a space does not back over the sidewalk (see Figure C-13). Vehicles should not block the sidewalk when parked in driveway.
Sidewalk and Driveway Intersections

Driveways crossing a sidewalk should be designed so that both pedestrians and drivers are able to negotiate the sidewalk-driveway crossing efficiently and safely. When the change in cross slope is too severe, one wheel of a wheelchair or one leg of a walker may lose contact with the ground. Pedestrians are also more prone to stumble on surfaces with rapidly changing cross slopes. For this reason, the maximum cross-slope at any point on a sidewalk (including the crossing of a driveway) is two percent (2%). Wherever possible the sidewalk should be carried across the driveway without a change with respect to the normal sidewalk profile. When the sidewalk abuts the back of the curb, a “walk-around” (see Figure C-14) should be considered. This design transitions the sidewalk laterally to provide greater distance between the flow line of the gutter and the sidewalk. This allows the sidewalk to remain at normal elevation without requiring an excessive driveway slope. The “walk around” design may not be possible if there is insufficient right-of-way available. In this case, the sidewalk grade must be lowered but preferably not all the way to street grade so that drainage in the gutter is maintained.
Figure C-14. Illustration of a “Walk-Around” Design
Section 7 — Visibility

Drivers must be able to locate a driveway in time to reduce speed and negotiate the entry maneuver. Signing and lighting can be used to provide drivers with information regarding driveway opening locations a considerable distance in advance. On divided driveways, the sign should be located within the divider separating the entrance and exit sides of the driveway. Lighting can illuminate the junction of the driveway and the highway.
Section 8 — References


Appendix D — Right Turn Slip Lane Design Guidelines

Contents:

Section 1 — Introduction
Section 2 — New Construction
Section 3 — Retrofitting Treatments
Section 4 — References
Section 1 — Introduction

Right-turn slip lanes are advantageous to motorists as they reduce delays by separating right-turning traffic from through lanes and allow higher-speed right turns. The right-turn slip lane channelizing island can also provide a refuge area for crossing pedestrians, reducing their exposure by allowing them to cross the roadway in two stages. Providing safe mobility for pedestrians and cyclists while reducing delays for motorists are potentially conflicting objectives. Accordingly, right-turn slip lane designs should create a balance between the safety and mobility of all roadway users.

This Appendix provides guidance on the design of right-turn slip lanes, including lane and raised island geometric layouts, pavement marking guidelines, pedestrian and bikeway guidelines, and accommodations for pedestrians with disabilities. Guidance is provided for urban, suburban, and rural roadway environments. Common issues at existing right-turn slip lanes have also been identified and treatment options for retrofitting are provided.
Urban Design

Right-turn slip lanes at urban intersections shall be designed to accommodate both motorists and pedestrians. Considerations should be given for crosswalk compliance by motorists and pedestrians, design vehicle accommodations, speed of turning traffic, provisions of auxiliary lanes, adjacent land uses, visibility of anticipated pedestrian traffic, and acuity of the cross-street traffic from the right turn drivers’ perspective. In areas where pedestrian activity is moderate to high, raised crosswalks may be installed to slow turning motorists and improve their likelihood of yielding to crossing pedestrians. A raised crosswalk may in some instances have the benefit of reducing the length of a required ramp on the pedestrian island. However, raised crosswalks are not recommended along high-speed facilities.

The following recommendations address these considerations and are reflected in the configurations shown in Figure D-4 (with a deceleration lane) and D-5 (without a deceleration lane).

1. **Angle of Entry:** The angle of entry between the slip lane and the cross street is recommended to be 70 degrees. This configuration slows motorists, reduces the head-turning movement to look for gaps in oncoming traffic, and makes it easier for motorists to identify crossing pedestrians. If an angle of 70 degrees is not achievable due to constraints, reduce accordingly. The minimum recommended angle is 55 degrees.

2. **Curb Radii and Curb-to-Curb (throat) Width:** The majority of traffic on urban streets is expected to be passenger cars and single unit trucks. However, to accommodate the turning movement of larger vehicles, the curb radius and curb-to-curb width can be designed for a larger design vehicle while striped to delineate the path for a smaller vehicle. For guidance on curb radii design for different vehicle classes, see the discussion in Chapter 3 on Urban Streets and the Intersections subsection, and in Minimum Designs for Truck and Bus Turns in Chapter 7. See Signage and Pavement Markings below for lane markings.

3. **Channelizing Island:** Refer to Chapter 7, Minimum Designs for Truck and Bus Turns - Channelization for proper size of channelizing islands. Islands are recommended to have a minimum side length of 15 ft, excluding the corner radii, as discussed in AASHTO’s A Policy on the Geometric Design of Highways and Streets. Channelizing islands should be offset from the edge of the traveled way to reduce their vulnerability. A 12 ft side length may be used in special circumstances, where the 15 ft minimum can’t be met due to highly constrained conditions. See Figure D-1 for design guidance on curb offset and tapering. In the presence of a bicycle lane, which serves as a separation between the curb and the travel lane, curbs need not be offset. Additional information on appropriate curb type and design can be found in Basic Design Criteria, Chapter 2, Section 6, and in Chapter 7, Minimum Designs for Truck and Bus Turns.
Figure D-1. Details for Channelizing Island Design for Right-Turn Slip Lane
Pedestrian accommodations are a central component of the design. The pedestrian access routes across channelizing islands are typically set at the top of curb elevation using a series of curb ramps and landings (Figure D-2-A). Smaller channelizing islands may not have adequate space to provide the necessary curb ramps, so the pedestrian access route can be cut through the island flush with the gutter grades of the roadway (Figure D-2-B). This option may be easier for some pedestrians to navigate and does provide some wayfinding benefits, but can also collect water and debris increasing maintenance needs. A third option is to raise the accessible route to at least 2 in above roadway gutter grade (Figure D-2-C). This solution helps reduce the maintenance required for cut-through islands and still provides some wayfinding clues. If a raised crosswalk is used this would also result in a reduction in the length of the needed ramp for the options depicted in Figures D-2-A, and D-2-C. Provide flared sides where the pedestrian circulation path crosses the curb ramp. Flared sides shall be sloped at 10% maximum, measured parallel to the curb. Curb returns may be used only where pedestrians would not normally walk across the ramp, either 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.
Figure D-2. Combination Island Ramps (Per Pedestrian Facilities Curb Ramp Standards)
If partial cut-through sections are used, a 10 percent flare and rounded corners along the pedestrian pathway should be used to provide better wheelchair mobility.

4. **Deceleration Lane:** These lanes allow motorists to decrease speed before negotiating a turn while separated from through traffic. This separation helps pedestrians identify right-turning vehicles. See *Number, Location, and Spacing of Access Connections* in Chapter 2 of the TxDOT Access Management Manual for volume thresholds for installing deceleration lanes. Refer to *Urban Streets* in Chapter 3 for design recommendations for deceleration lanes. In the event that conditions do not necessitate a deceleration lane or right-of-way is restricted, consideration should be given to using a taper, as defined in Chapter 3. See Figures D-4 and D-5 for sample right-turn slip lane designs with and without a deceleration lane.

5. **Acceleration Lane:** Acceleration lanes typically are not used on urban streets since they make it more difficult for pedestrians, especially the visually impaired, to cross the turning roadway. Accordingly, acceleration lanes are not advisable where pedestrian activity is anticipated.

6. **Drainage:** Any necessary inlets should be designed and placed on the upstream side of the crosswalk at a location that prevents, or limits to the extent practical, the spread of water into the crosswalk. Cut-through access should be situated as to minimize paths for water flow. Avoid placing drainage low points at or near the ADA curb ramps.

7. **Lighting:** Intersections with channelization should be illuminated. Lighting helps motorists identify islands, diverge and merge locations, turning roadways, and pedestrian crossings. Adequate lighting at urban intersections, including illumination of crossing locations, is important, particularly where pedestrian activity is expected at night.

8. **Apparatus and Pole Placement:** ITS equipment, signal and utility poles and apparatuses should be outside of paved pedestrian walkways and landing areas. Refer to the TMUTCD and TxDOT standard drawings for guidance on mounting heights and limits on object protrusion into pedestrian facilities. When pedestrian facilities are not initially installed, care should be taken to avoid the placement of apparatuses and poles in anticipated or planned locations of future pedestrian walkways and landing areas.

9. **Crosswalk Location:** Crosswalks should be placed toward the middle of the channelized island with a minimum of 20 ft between crosswalk and yield line for intersecting street Crosswalks may be placed near the beginning of the channelized island if conditions do not permit a centralized location or it is more conducive to the natural pathway of pedestrians. When the crossing is located at the beginning of the channelized island, care should be taken to place it such that there is enough space available at the ramp location for an appropriate landing area. Placement of the crosswalk near the end of the turning roadway is not recommended as motorists are expected to encroach on the crosswalk as they yield to oncoming traffic. Also, motorists arriving at the downstream end of the turning roadway typically focus their attention on cross street traffic rather than crossing pedestrians.

10. **Crosswalk Orientation:** The pedestrian crosswalk should be oriented perpendicular to the turning roadway to shorten the crossing distance for pedestrians and to place approaching vehicles in the periphery of pedestrians.
11. **Crosswalk Markings**: At locations where pedestrian activity is anticipated, longitudinal “ladder” markings are recommended to delineate the crossing location. The transverse markings facilitate wayfinding for visually impaired pedestrians and the inclusion of longitudinal lines provides additional visibility for approaching motorists. Alternatively, longitudinal markings alone can be installed to define the crossing path. Refer to the *TMUTCD* for further guidance on the installation of crosswalk markings.

12. **Signage and Pavement Markings**: Yield signs are typically the appropriate control devices for right-turn slip lanes at urban intersections. The yield line is used alongside the yield sign to draw attention to the need to yield to cross-street traffic. Refer to the *TMUTCD* and TxDOT standard drawings for guidance on yield sign and yield line placement. Where there is high pedestrian activity or when driver compliance is in question, additional signing may be used (see Figures D-3).

![Figure D-3. Supplemental Crosswalk Signs, W11-2 and W16-7PL Sign and Plaque (TMUTCD)](image)

The travel lane should be striped to a minimum of 10 ft in width (11 ft typical) to accommodate a passenger vehicle, and the void area (shoulder) may be delineated by diagonal lines as shown in Figures D-4 and D-5. A raised truck apron may also be considered in the void area to further enhance the channelization of passenger vehicles.

13. **Bicycle Lane**: When a bicycle lane is used it should be striped appropriately to define right-of-way and shared spaces as discussed in the AASHTO *Guide for the Development of Bicycle Facilities*, as shown in the *TMUTCD*, and as detailed on TxDOT standard drawings for bicycle lanes. Bicyclists intending to make a right turn can use the right-turn lane/turning roadway and operate like a motorized vehicle.
Figure D-4. Right-Turn Slip Lane Design for Urban Intersections with Deceleration Lane
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.

Figure D-5. Right-Turn Slip Lane Design for Urban Intersections without Deceleration Lane
1. **Angle of Entry**: See guidance provided under *Urban Design, Angle of Entry*.

2. **Curb Radii and Curb-to-Curb (Throat) Width**: The radii and throat width for right-turning roadways in suburban areas should be designed to accommodate larger design vehicles. In the event that the area becomes more urbanized in the future, the turning roadway can be striped to delineate a tighter radius, promoting lower speeds and improving visibility of pedestrians for motorists (see Figures D-4 and D-5). See *Cross Sectional Elements* in Chapter 2 for striped lane width.

3. **Channelizing Island**: See guidance provided under *Urban Design, Channelizing Island*.

4. **Deceleration Lane**: See guidance provided under *Urban Design, Deceleration Lane*.

5. **Acceleration Lane**: Acceleration lanes may be used; however, they make it more difficult for pedestrians, especially those with visual impairments, to cross the turning roadway. Therefore, acceleration lanes are not advisable where pedestrian activity is anticipated.

6. **Drainage**: See guidance provided under *Urban Design, Drainage*. When pedestrian facilities are not initially installed, consideration should be given to determine where crosswalks may be installed in the future to avoid conflicts with inlet locations.

7. **Lighting**: See guidance provided under *Urban Design, Lighting*.

8. **Apparatus and Pole Placement**: See guidance provided under *Urban Design, Apparatus and Pole Placement*. When pedestrian facilities are not initially installed, care should be taken when placing ITS equipment, apparatuses and poles to avoid the anticipated or planned location of future pedestrian walkways and landing areas.

9. **Crosswalk Location**: See guidance provided under *Urban Design, Crosswalk Location*. In the event that an acceleration lane is present, consider placing the crosswalk at the upstream end of the turning roadway, placing pedestrians in the line of sight for drivers as they decelerate to make the turn. At the downstream end, on approach to an acceleration lane, drivers will be accelerating out of the turn and more likely be focusing on cross-street traffic.

10. **Crosswalk Orientation**: See guidance provided under *Urban Design, Crosswalk Orientation*.

11. **Crosswalk Markings**: See guidance provided under *Urban Design, Crosswalk Markings*.

12. **Signage**: See guidance provided under *Urban Design, Signage*.

13. **Bicycle Lane**: See guidance provided under *Urban Design, Bike Lane*.

**Rural Design**

If pedestrian activity is expected, see the guidelines for suburban design section.

If pedestrians are not an issue, the following guidelines may be used:
1. **Angle of Entry:** In rural areas, the angle of entry between the slip lane and the cross street is typically flatter than in urban areas to facilitate high-speed turns and the channelizing island should be constructed as an isosceles triangle.

2. **Radius:** For guidance on radius design for different vehicle classes, see *Minimum Designs for Truck and Bus Turns* in Chapter 7. Superelevation should be provided in rural right turn slip bays. Therefore, radii may be governed by the design speed.

3. **Channelizing Island:** The channelizing island may be flush with the pavement or depressed. Careful consideration should be made in rural areas for the use of curbed islands, particularly along high-speed facilities and at isolated intersections. If curbs are installed, they should be sloped and offset from the traveled way, and islands made clearly visible to motorists. See Chapter 9 of AASHTO’s *A Policy on Geometric Design of Highways and Streets* for design guidance on curb offset and tapering (Figure D-1). Details on island approach treatment and delineation are presented in Chapter 3 of the *Texas Manual on Uniform Traffic Control Devices (TMUTCD)* and Chapter 9 of AASHTO’s *A Policy on Geometric Design of Highways and Streets*.


5. **Acceleration Lane:** See Chapter 2, *Number, Location, and Spacing of Access Connections* of the TxDOT *Access Management Manual* for volume thresholds for installing acceleration lanes. These lanes provide a benefit when right-turn volumes are especially high and/or the speed differential between turning vehicles and vehicles on the cross street is large. Acceleration lanes provide benefits to motorists by allowing them to reach a higher speed prior to merging, but may increase sideswipe crashes. Acceleration lanes have been found to be preferred by elderly drivers at high-speed intersection locations. See Chapter 3, *Two-Lane Rural Highways, Speed Change Lanes, Right Turn Acceleration Lanes* and *Multi-Lane Rural Highways, Turn Lanes, Acceleration Lanes* of this manual for design recommendations for acceleration lanes for two-lane and multilane rural highways, respectively.

6. **Lighting:** Intersections with channelization should be illuminated.
Section 3 — Retrofitting Treatments

Common issues encountered at right-turn slip lanes include the absence of adequate refuge in the channelizing island for crossing pedestrians, failure of motorists to yield to crossing pedestrians, pedestrian noncompliance with the crosswalk location, high-speed turns jeopardizing pedestrian safety, low visibility of crossing pedestrians, and excessive head turning required to observe oncoming traffic. These observations should be supported by collision diagrams and/or crash analyses of the intersection. Potential retrofitting treatments designed to mitigate these issues are presented below.

Providing Proper Refuge for Pedestrians

Where the channelizing island along a right-turn slip lane is painted and does not provide adequate refuge for crossing pedestrians, consideration should be given to installing a raised island. Raised pedestrian islands reduce the crossing distance for pedestrians by allowing pedestrians to cross the through lanes and turning roadway separately while taking refuge on the island between. The reduction in crossing distance may also improve signal timing.

At intersections where there is a raised channelizing island but it is not large enough to provide refuge for pedestrians, the island should be expanded to establish an adequate landing area for pedestrians and comply with ADA regulations. The minimum recommended size of the channelizing island is 300 ft² for intersections in all area types; the specific site conditions will dictate the final size.

Yielding to Crossing Pedestrians

At intersections where there is a concern that motorists are failing to yield to crossing pedestrians, several treatments to improve compliance can be considered. These treatments are intended to achieve one or both of the following: improve the visibility of the crosswalk, and decrease the speed of right-turning motorists. When visibility is a concern and the crossing is currently marked, consideration may be given to upgrading the crosswalk markings to include longitudinal bars and incorporate a “ladder” pattern. An advanced warning sign (W11-2) and yield stripe may also be installed (see Figure D-3).

In areas where pedestrian activity is moderate to high, raised crosswalks may be installed to slow turning motorists and improve their likelihood of yielding to crossing pedestrians. However, raised crosswalks are not recommended along high-speed facilities. Signs with pedestrian-actuated flashing beacons may be installed to provide an advanced warning to approaching motorists of the need to comply with the crossing location. Caution must be taken when installing beacons where pedestrian activity is minimal and the infrequent activation of these beacons may violate driver expectations.
NCHRP Report 562 provides additional guidance concerning different types of crossing treatments based on observed conditions, including thresholds for pedestrian and vehicle volumes and roadway speed.

Crosswalk Location

Crosswalks should be placed in the middle of the channelized roadway, perpendicular to the direction of traffic. Being the shortest path, this treatment is likely to increase compliance. Signs may be used to direct pedestrians to the location where they are expected to cross. The R9-2 sign (Figure D-6) is a regulatory sign for crossing pedestrians.

Reducing Speeds in the Channelized Roadway

High-speed turns are generally promoted by wide, sweeping turning roadways and the presence of acceleration lanes downstream of the right-turn slip lane. When applicable, consideration should be given to striping turning roadways in order to delineate the path for passenger vehicles and promote a sharper entry angle with the cross street. Also, the presence of a deceleration lane upstream of the turning roadway provides an area for approaching vehicles to decrease speed before making the turn while separated from through traffic. Consideration for removing the acceleration lane where their presence is not necessary (mainly along urban and suburban streets) may be appropriate as they promote high-speed turns and may cause inconsistent driver behavior (e.g., some drivers may stop or slow to look for oncoming traffic before they proceed, while others continue at pace into the acceleration lane and look for a gap closer to the downstream merge location). See Figure D-4 for the recommended design configuration of a right-turn slip lane with a deceleration lane, including pavement markings.
Section 3 — Retrofitting Treatments

Enhancing Visibility of Crossing Pedestrians

At locations where the visibility of pedestrians is low, warning signs may be installed in advance of the crosswalk to alert motorists of the presence of a crosswalk ahead. Consideration should be given to striping the crosswalk with “ladder” markings to enhance the visibility of the crossing location. Provision of a deceleration lane upstream of the turning roadway better accommodates a decrease in speed by approaching motorists, which provides them more time to spot crossing pedestrians. Reconstructing the turning roadway and channelizing island to incorporate a more pedestrian-friendly design may be an option as part of intersection improvements. If intersection lighting is absent or insufficient, addition or enhancement of lighting to illuminate the crossing and surrounding area may be appropriate. Other potential treatments include rectangular rapid flashing beacons, or other pedestrian-actuated traffic control devices that alert motorists to the presence of the crossing location only when pedestrians are present. This not only improves the safety conditions at these intersections but actuation may reduce the impact to motorists’ mobility.

Reducing Head Turning to Spot Oncoming Traffic

At intersections where motorists are required to turn their heads excessively to observe oncoming traffic, consideration should be given to reconfiguring the channelizing island and turning roadway such that the angle of entry is closer to 70 degrees. As a result, navigating the turning roadway does not require as much physical effort to observe cross street traffic. This may involve reconstructing the channelizing island/outside curb radius or restriping the island area and turning path.

Slip Lane Removal

If none of the available right-turn slip lane treatments will address existing safety problems at the turning roadway, and pedestrian activity is very high, consideration may be made to close the slip lane and transform the area into a pedestrian-friendly corner with street furniture, benches, and landscaping. A shared through-right-turn lane would replace the slip lane to accommodate the right turning movement. However, this option should be carefully considered as the removal of the slip lane may eliminate a number of benefits, including the reduction of vehicular delays and rear-end crashes.
Section 4 — References


9. Pedestrian Refuge Island, Metropolitan Transportation Commission.


Appendix E — Alternative Intersections and Interchanges

Contents:

Section 1 — Overview
Section 2 — Roundabouts
Section 3 — Diverging Diamond Interchange (DDI)
Section 4 — Median U-Turn Intersection (MUT)
Section 5 — Restricted Crossing U-Turn Intersection (RCUT)
Section 6 — Displaced Left Turn Intersection (DLT)
Section 7 — References
Section 1 — Overview

Introduction: Alternative Intersections and Interchanges

In recent years, the number of alternative intersections and interchanges has increased substantially in the United States and in Texas. As these numbers have increased, the body of data documenting the efficacy and appropriate applications of these intersections has also increased. Some of the advantages of these types of intersections/interchanges include the following:

- An improvement in traffic flow by eliminating, relocating or modifying conflict points
- Improved signal phasing and operations
- A general decrease in crashes, and particularly a reduction in more severe crashes
- A decrease in congestion and a reduction in traffic bottlenecks
- Improvement of intersection delay, Level of Service (LOS), travel time, and vehicle throughput
- The increased ability to maintain existing bridge structures
- A possible reduction in the amount of ROW required for new projects

There are many tools currently available to conduct a preliminary (Stage 1) assessment to determine if a particular alternative is viable and preferable. The FHWA Alternative Intersections/Interchanges Informational Report is a report that provides guidance on proper selection, and the FHWA Capacity Analysis for Planning of Junctions (CAP-X) is an Excel based program that can be used to evaluate selected types of innovative junction designs (eight intersections, five interchanges, three roundabouts and two mini-roundabouts) using given peak flow volumes. Additionally, the FHWA Safety Performance Intersection Control Evaluations (SPICE), an excel based program uses safety performance functions (SPFs) in Part C of the Highway Safety Manual (HSM) to evaluate and compare the safety aspects of at-grade intersection alternatives. The SPICE tool allows the selection of default safety performance functions (SPFs) and high quality crash modification factors (CMFs) from Part D of the HSM and CMF Clearinghouse to predict crash frequency and severity of intersection control strategies.

Stage 2 analysis would include a more detailed analysis of the preferred alternatives from Stage 1. These could include use of the Highway Safety Manual (HSM) procedures and associated programs such as Interactive Highway Safety Design Model (IHSDM), and Safety Analyst for safety analysis; Highway Capacity Manual (HCM) based tools such as Synchro, Sidra, and microsimulation modeling tools such as Vissim for traffic operational analysis; and cost-benefit analysis models.

The Design and Traffic Safety Divisions will be developing suggested procedures and protocols that incorporate Intersection Control Evaluation (ICE) which is a data-driven, performance-based
framework and approach to objectively screen intersection alternatives and identify an optimal geometric and control solution for an intersection.

Early in the selection process of an alternative intersection, public involvement should be conducted to educate and allow feedback from the stakeholder and local community concerning the benefits and proper function of the chosen alternative, including vehicular, pedestrian, and bicyclist accommodations.
**Section 2 — Roundabouts**

**Overview**

A roundabout is a form of a circular intersection in which traffic travels counterclockwise around a central island and entering traffic yields to the circulating traffic. Roundabouts have been demonstrated to significantly reduce the number of severe crashes at intersections, improve Level of Service (LOS), and increase capacity. TxDOT has adopted *NCHRP Report 672 (Roundabouts an Informational Guide, 2nd Edition)* as the primary source for roundabout design guidelines. The information contained in this Appendix is considered a companion guide to *NCHRP Report 672*, and is intended to document TxDOT’s suggested approach to roundabout design.

**Planning**

During the planning stage many considerations are needed to determine the applicability of a roundabout, including ROW, utilities, access management, operations of adjacent intersections, safety impacts, existing and predicted future traffic volume, and driver education. Locations that meet or nearly meet signal warrants should be given consideration for roundabout installation. Intersections that are, or proposed to be, all-way stop control may also be good candidates for a roundabout. Figures E-1 through E-4 show the design characteristics and features of the three roundabout categories: Mini-roundabout, Single Lane Roundabout, and Multilane Roundabout.

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Mini-Roundabout</th>
<th>Single-Lane Roundabout</th>
<th>Multilane Roundabout</th>
</tr>
</thead>
<tbody>
<tr>
<td>Desirable maximum entry design speed</td>
<td>15 to 20 mph</td>
<td>20 to 25 mph</td>
<td>25 to 30 mph</td>
</tr>
<tr>
<td></td>
<td>(25 to 30 km/h)</td>
<td>(30 to 40 km/h)</td>
<td>(40 to 50 km/h)</td>
</tr>
<tr>
<td>Maximum number of entering lanes per approach</td>
<td>1</td>
<td>1</td>
<td>2+</td>
</tr>
<tr>
<td>Typical designed circle diameter</td>
<td>45 to 90 ft</td>
<td>90 to 180 ft</td>
<td>150 to 300 ft</td>
</tr>
<tr>
<td></td>
<td>(13 to 27 m)</td>
<td>(27 to 55 m)</td>
<td>(46 to 91 m)</td>
</tr>
<tr>
<td>Central island treatment</td>
<td>Fully traversable</td>
<td>Raised (may have traversable apron)</td>
<td>Raised (may have traversable apron)</td>
</tr>
<tr>
<td>Typical service volumes on 4-leg roundabout below which may be expected to operate without requiring a detailed capacity analysis (veh/day)*</td>
<td>Up to approximately 15,000</td>
<td>Up to approximately 25,000</td>
<td>Up to approximately 45,000 for two-lane roundabout</td>
</tr>
</tbody>
</table>

*Operational analysis needed to verify upper limit for specific applications or for roundabouts with more than two lanes or four legs.

*Source: NCHRP Report 672 - Exhibit I-9*

**Figure E-1. Design Characteristics of Three Roundabout Categories**
Figure E-2. Features of a Typical Mini-Roundabout

Figure E-3. Features of a Typical Single-lane Roundabout
The preference in Texas is to utilize a single lane roundabout as long as possible to avoid unnecessarily overbuilding the roundabout. Experience shows that most roundabouts that are redesigned are actually reduced in size (e.g., 2 to 1 lane). In some instances, the reduction of lanes was done to diminish minor crash patterns. If a single lane roundabout is found to be adequate for up to 10 years after the opening year, a single lane roundabout should be constructed. If a multilane roundabout is required before the design year (20 years after opening year), the single lane roundabout should be constructed having the footprint of a multilane roundabout and be designed to be easily retrofitted to a multilane roundabout when needed. For multi-lane roundabout designs, additional independent reviews are recommended by designers with expertise in roundabout design.

**Geometric Design**

The primary goals of any roundabout design are to provide:

- Slow entry and consistent speeds throughout the roundabout by using deflection.
- The appropriate number of lanes and lane assignment to achieve capacity, lane volume balance, and lane continuity.
- Smooth channelization that result in vehicles naturally using the intended lanes.
- Adequate accommodation for the design vehicles.
- Safe accommodations for pedestrians and bicyclists.
No more than the minimum intersection sight distance for driver recognition of the intersection and conflicting users.

The two most relevant aspects of sight distance for roundabouts are stopping sight distance and intersection sight distance. Stopping sight distance should be provided at every point within a roundabout, and on each entering and exiting approach. At roundabouts, the only locations requiring evaluation of intersection sight distances are the entries. A combined sight distance diagram should be overlaid onto a single drawing to illustrate the clear vision areas for the intersection. Chapter 6 of NCHRP 672 provides specific guidance with respect to sight distance determination and application.

The design vehicle is the largest vehicle likely to use the intersection; this is typically a WB-67 vehicle in Texas (a smaller design vehicle may be used for urban/local or suburban classifications). The anticipated truck movements through the roundabout should be determined and AutoTURN, AutoTrack, (or a similar tool) used to determine vehicle encroachment within the roundabout. Currently there is research underway to update the vehicle turning movement templates since they are decades old and tend to be overly conservative. If a significant number of large trucks is anticipated through an intersection, it is advisable to find an area where the wheel paths of the respective design vehicle could be physically tested and documented. The largest truck movements may be accommodated by using a truck apron (with a roll down curb) within the central island. The apron provides additional paved surface for the design vehicle while keeping the actual circulatory roadway width narrow enough to maintain speed control for smaller passenger cars. The basic geometric elements of a roundabout and typical inscribed circle diameters are shown in Figures E-5 and E-6:
Figure E-5. Basic Geometric Elements of a Roundabout

<table>
<thead>
<tr>
<th>Roundabout Configuration</th>
<th>Typical Design Vehicle</th>
<th>Common Inscribed Circle Diameter Range*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mini-Roundabout</td>
<td>SU-30 (SU-9)</td>
<td>45 to 90 ft (14 to 27 m)</td>
</tr>
<tr>
<td>Single-Lane Roundabout</td>
<td>B-40 (B-12)</td>
<td>90 to 150 ft (27 to 46 m)</td>
</tr>
<tr>
<td></td>
<td>WB-50 (WB-15)</td>
<td>105 to 150 ft (32 to 46 m)</td>
</tr>
<tr>
<td></td>
<td>WB-67 (WB-20)</td>
<td>130 to 180 ft (40 to 55 m)</td>
</tr>
<tr>
<td>Multilane Roundabout (2 lanes)</td>
<td>WB-50 (WB-15)</td>
<td>150 to 220 ft (46 to 67 m)</td>
</tr>
<tr>
<td></td>
<td>WB-67 (WB-20)</td>
<td>165 to 220 ft (50 to 67 m)</td>
</tr>
<tr>
<td>Multilane Roundabout (3 lanes)</td>
<td>WB-50 (WB-15)</td>
<td>200 to 250 ft (61 to 76 m)</td>
</tr>
<tr>
<td></td>
<td>WB-67 (WB-20)</td>
<td>220 to 300 ft (67 to 91 m)</td>
</tr>
</tbody>
</table>

* Assumes 90° angles between entries and no more than four legs. List of possible design vehicles is not all-inclusive.

Source: NCHRP Report 672 - Exhibit 6-9

Figure E-6. Typical Inscribed Circle Diameter Ranges
Entry Width

The entry width is determined by the design vehicle entering the roundabout. For single-lane entrances a typical range for entry width would be from 16 to 20 ft. A 17 ft entry width would be considered a good starting point. Entry widths greater than 20 ft often confuse drivers into thinking there are two lanes available when there is only one circulatory lane in the roundabout. Chapter 6 of NCHRP Report 672 provides additional information on entry widths for roundabouts.

Circulatory Roadway Width

The circulatory width is generated from the number of lanes entering the roundabout and the turning movements of the design vehicle. For single-lane roundabouts, a 18 to 20 ft circulatory roadway width would be typical. For two-lane roundabouts, it may be necessary to accommodate the turning movements for two passenger vehicles side by side or, assuming infrequent traffic from semi-trailers, a passenger vehicle and single unit truck side by side. Where semi-trailer traffic is greater than 10 percent of the overall traffic, it may be sufficient to meet the turning movements for a semi-trailer in conjunction with a passenger vehicle or single unit truck. Chapter 6 of NCHRP Report 672 provides additional information on circulatory roadway widths for roundabouts.

Entry Curve

Capacity and safety are both impacted by the determination of the entry curve radius. Large entry curves may generate relatively fast entry speeds, and reductions in capacity are generally noticed with entry curves less than 50 ft. Entry curves with a radius of 50 to 100 ft are typical for an urban single-lane roundabout. Entry radii from 70 to 85 ft would be a good beginning point for determining the entry radius of a multilane roundabout. Chapter 6 of NCHRP Report 672 provides additional information on entry curves for roundabouts.

Exit Curve

The exit curve design is controlled by conditions such as pedestrian traffic, whether it is an urban roundabout or a rural roundabout, and geometric limitations. In order to avoid congestion and crashes at the exit point in a roundabout, exit curve radii are typically greater than an entry curve radii. An exit curve radii of 200 to 400 ft is recommended to promote proper tangent alignment from the circulatory to the exit. Larger trucks are more easily accommodated by larger exit curve radii.

Roundabout Speeds

The most critical design objective for a roundabout is to maintain low and consistent speeds at the entry and throughout the roundabout. Horizontal curvature and reduced pavement widths influence low speed conditions in roundabouts. The maximum entering design speeds based on a theoretical
fastest path are: Mini-Roundabout – 20 mph; Single Lane – 25 mph; Multilane – 25 to 30 mph. See Figure E-7 and Figure E-8 for depictions of the fastest vehicle path through a single lane-lane roundabout and a multilane roundabout, respectively. See Chapter 6 of NCHRP Report 672 for recommended horizontal curvatures and pavement widths to produce proper speeds.

Figure E-7. Fastest Vehicle Path through Single-lane Roundabout

Figure E-8. Fastest Vehicle Path through Multilane Roundabout
See NCHRP Report 672 for additional discussion on roundabout geometric guidance.

**Vertical Geometry**

In forming the vertical geometry for a roundabout, a thorough investigation of the approach roadways and central island profile is needed. The approach roadway profiles should connect to the profile for the circumference of the central island. The vertical geometry for the circumference of the central island is typically a sine curve. Examples of vertical geometry for roundabouts are shown in Figures E-9 and E-10. Note that cross slopes and/or truck apron slopes may be less than those shown in the examples to further accommodate low boy type vehicles.

*Figure E-9. Sample Central Island Profile*
Figure E-10. Sample Central Island Profile

NOTE: All grades shown are examples.
Cross-Slope

The typical cross-slope for a roundabout is an outward 2 percent slope from the central island. Note that the cross slope may be less than 2% to further accommodate low boy type vehicles.

Splitter Islands

AASHTO standard guidelines govern the design of the splitter island. See Figure E-11 for minimum values for nose radii and offsets. Note that larger radii than those specified in the minimums are recommended in order to avoid the breaking off of the curbs.

![Figure E-11. Minimum Splitter Island Nose Radii and Offsets](image)

Pedestrian and Bicyclist Considerations

The design of a roundabout allows pedestrians to cross one direction of traffic at a time on each leg of the roundabout. The reduction in both vehicle speeds and conflict points enhances pedestrian safety. By providing refuge on the splitter island, the pedestrians cross only one direction of conflicting traffic at a time. The minimum dimensions of the splitter islands are shown in Figure E-12 (Note that the 6 ft measured in Detail “A” is from BOC to BOC). Wherever possible, sidewalks at roundabouts should be set back from the edge of the circulatory roadway with a landscape strip. Chapter 6 of NCHRP Report 672 provides additional specific guidance with respect to pedestrian design at roundabouts.
Roundabouts also provide additional safety for bicyclists due to the slower vehicular speeds which result in less of a speed differential between bicyclists and motor vehicles since typical on-road bicycle speeds are between 10 and 20 mph. Bicyclists will typically take a full lane to traverse the roundabout. Less experienced bicyclists may also have the option of choosing to travel on the sidewalks provided for pedestrians and bicyclists at the roundabout. In these instances, slip ramps (which is a type of ramp for getting bicyclists from the roadway onto the sidewalk) should be considered. Chapter 6 of *NCHRP Report 672* provides additional specific guidance with respect to bicycle design at roundabouts.

**Access Management**

Access management is central to the proper functioning of a roundabout from an operational and safety standpoint. Avoid locating driveways with direct access to a roundabout. If unavoidable, *NCHRP Report 672* provides guidance on situations where driveways may be allowed direct access. Additionally, access points near a roundabout have restricted operations due to the channelization of the roundabout. The ability to provide full access is governed by the following factors:

- The capacity of the minor movements at the access point.
The need to provide left turn storage downstream of the roundabout on the major street to serve the access point in order to prevent blocking the major street flow.

The available space between the access point and the roundabout in order to allow for adequate splitter island design and left-turn pocket channelization (see Figure E-13 for an example). Note the resulting storage and taper length should be analyzed using the anticipated turning volume into the driveway to avoid queue spillback into the roundabout.

Sight distance needs.

Sight distance needs. A driver at the access point should have proper intersection sight distance and should be visible when approaching or departing the roundabout, as applicable.

Source: NCHRP Report 672 - Exhibit 6-91

Figure E-13. Typical Dimensions for Left-turn Access near Roundabouts

Reference NCHRP Report 672 for additional guidance with respect to access management at roundabouts.
Section 3 — Diverging Diamond Interchange (DDI)

Overview

The DDI is an interchange form that allows the two directions of traffic on the crossroad to temporarily divide and cross to the opposite side to gain access to and from the freeway more easily. The primary difference between a DDI and a conventional interchange is the design of directional crossovers on either side of the interchange. This eliminates the need for left-turning vehicles to cross the path of approaching vehicles. By shifting cross street traffic to the left side of the street between the signalized intersections, vehicles on the crossroad making a left turn on to or off of ramps do not conflict with vehicles approaching from other directions. TxDOT has adopted the FHWA Diverging Diamond Interchange Informational Guide as the primary source for DDI guidelines. The information contained in this Appendix is considered a companion guide and is intended to document TxDOT’s suggested approach to DDI design.

Some of the documented benefits of a DDI include:

- Capacity improvements with two-phase signal configurations.
- Safety improvements due to a reduction in the number of conflict points.
- Possible lower costs due to a smaller footprint, shorter construction time, and the possibility of being able to salvage existing bridge structures.

Figure E-14 shows the design characteristics and key features of the DDI.

Figure E-14. Key Characteristics of a DDI

Source: FHWA DDI Informational Guide - Exhibit F-1
Design Considerations

Appropriate geometrics is central in the proper functioning of a DDI. The design vehicle is typically a WB-67 vehicle in Texas. The anticipated truck movements through the DDI should be determined and AutoTURN (or a similar tool) used to determine vehicle encroachments within the DDI. There is research underway to update the current turning movement templates since they are decades old and tend to be conservative. If a significant number of large trucks is anticipated through an intersection, it is advisable to find an area where the wheel paths of the respective design vehicle could be physically tested and documented. The largest truck movements may be accommodated by using a roll down curb in turning areas. The design speed of the DDI affects the reverse curve radii through the two intersection crossovers, and should typically range from 25 to 35 mph (see RDM Chapter 2, Section 4, Table 2-5 for corresponding curve radii values). The DDI profile should be relatively flat to increase driver sight distance. A tangent section is recommended before and after the crossovers to minimize the likelihood of wrong way maneuvers into opposing lanes, and the recommended crossover angle is 40-50 degrees or greater (see Figures E-15 and E-16 respectively). Figure E-17 depicts some typical curve radii ranges for lower-speed DDIs.

Source: PHWIDDI Informational Guide - Exhibit 7-16

Figure E-15. Tangent Length Approaching and Departing the Crossover
CROSSOVER GEOMETRY

- The greater the crossing angle, the less “different” the intersection will seem.
  - Recommended crossover angles of 40-50 degrees (or more)
  - Existing DDIs have angles as low as 28 degrees
  - Low crossover angles may increase the likelihood for wrong-way maneuvers into opposing lanes
  - Low angles increase crossing distances and increase signal clearance time

Source: FHWA Intersection and Interchange Geometrics (ITG) Workshop (May 2016)

Figure E-16. Crossover Geometry (Crossing Angle)

CROSSOVER GEOMETRY

Curve radii approaching and following the DDI crossover generally range from 150-300 feet

Source: FHWA Intersection and Interchange Geometrics (ITG) Workshop (May 2016)

Figure E-17. Crossover Geometry (Curve Radii)
Sight Distance

Drivers approaching or departing an intersection should have an unobstructed view of traffic control devices and sufficient length along the cross road to safely navigate the intersection. Insufficient sight distance is a significant factor in street crashes and near collisions. As with any other intersection, DDI intersections should provide stopping sight distance (SSD) and intersection sight distance (ISD). Sight distances should be checked for these conflict areas: walls, railings, tall landscaping, or other obstructions that may limit sight distance. Intersection sight distance may also be limited by barriers or other obstacles between the crossovers.

Horizontal Alignment Alternatives

There are three alignment alternatives resulting in a minimum cross-section along the cross road regardless of whether the facility is an over-or under pass. There are two types of alignments: the Symmetrical Alignment and the Shifted Alignment. Figure E-18 shows an example of symmetrical and shifted alignments. This results in distances of 600’ to 750’ between crossovers. If the distance between crossovers can be reduced it can boost traffic operations and limit the amount of right of way needed.

![Figure E-18. Alignment Alternatives](image-url)
Eliminating a few reverse curves can reduce the spacing between crossovers. Figure E-19 shows alignment alternatives where the number of reverse curves has been reduced with a resultant increase in median width. This results in distances of 400’ to 500’ between crossovers. Chapter 7 of the *FHWA Diverging Diamond Interchange Informational Guide*\(^6\) provides additional information on horizontal alignment options.

![Alignment Alternatives Diagram](image)

**Figure E-19. Reduced Reverse Curves in Alignment Alternatives**

**Auxiliary Lanes**

Auxiliary lanes would aid weaving traffic at a DDI interchange, smooth traffic flow, and provide added capacity. Figures E-20, E-21, E-22, and E-23 show some examples of auxiliary lanes at a DDI interchange. Chapter 7 of *FHWA Diverging Diamond Interchange Informational Guide*\(^6\) provides additional information on auxiliary lanes.
Figure E-20. Auxiliary Left Turn Lane Between Crossovers (Not Preferred)

Figure E-21. Auxiliary Left Turn Lane Developed Prior to the First Crossover

Figure E-22. Auxiliary Lane, Shared Left and Through, Developed Prior to the First Crossover
Pedestrian and Bicyclist Considerations

DDIs have many benefits for pedestrians which include: Allowing more crossing time per phase due to the two-phase signal operations, crossing only one direction of traffic resulting in reduced conflicts, and fewer travel lanes for a pedestrian to cross.

DDIs provide the option to direct pedestrians to either the outside of the intersection or to a center walkway. The wide area between opposing traffic allows the opportunity for a large sidewalk down the center that can be bordered by concrete barrier to allow additional protection and channelization for the pedestrians. Pedestrian facilities on the inside minimize conflicts with traffic turning left to and from the freeway, and allow the crossing of the interchange in all directions. When placing the pedestrian sidewalk along the outside some important considerations are: The location of the crosswalk with respect to the bridge structure or other sight obstructions to maintain good visibility for both the pedestrians and vehicles, and considering the turning radii to reduce speeds in the vicinity of pedestrians. Figures E-24 and E-25 depict the inside and outside pedestrian sidewalk options, respectively.
Figure E-24. Pedestrian-focused DDI – Center Walkway

Source: FHWA DDI Informational Guide - Exhibit 3-9
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 3 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 – 7 ft. are recommended through the DDI. (See Figure E-26)
- A separated sidewalk or wider shared-use path. This would typically entail the bicyclist disembarking on the upstream end of the DDI and then proceeding through the DDI in the same area designated for pedestrians.
- Shared-lane or on-street bicycle accommodations. Shared-lane (sharrow) markings may be used to reinforce to drivers that bicyclists are legal road users.

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.
From an access management standpoint, a DDI intersection provides full access control through the interchange. A traffic simulation should be conducted to determine the impacts and any needed mitigation for adjacent and nearby intersections in the corridor. Some of the possible disadvantages of a DDI include the following:

- Will not allow exit ramp to entrance ramp movements.
- Through movements along the frontage road can only be accommodated via bypass lanes or collector-distributor systems.
- May require modifications to nearby signalized intersections.
- Additional access management control beyond the interchange may be needed to prevent weaving maneuvers.
- May require the relocating or removal of adjacent streets/driveways to accommodate crossover and reverse curves.
Removes driveway access for corner development.

Reference the *FHWA Alternative Intersections/Interchanges Informational Report (AIIR)* for additional guidance with respect to access management.
Section 4 — Median U-Turn Intersection (MUT)

Overview

The MUT is an intersection that replaces direct left turns at an intersection with indirect left turns using a U-turn movement in a wide median. The MUT intersection eliminates left turns on both intersecting streets, thus reducing the number of traffic signal phases and conflict points at the main crossing intersection, which results in improved intersection operations and safety. TxDOT has adopted the *FHWA Median U-Turn Informational Guide* as the primary source for MUT Guidelines. The information contained in this Appendix is considered a companion guide and is intended to document TxDOT’s suggested approach to MUT design. See Figure E-27 for an example MUT configuration.

![Diagram of MUT Intersection](image)

*Source: FHWA MEDIAN U-TURN INFORMATIONAL GUIDE - EXHIBIT 11*

**Figure E-27. Example of a MUT Intersection With One Signal in Main Intersection**

Some of the documented benefits of a MUT include:

- Removal of left-turn signal phasing which allows the intersection to operate well with a shorter cycle length.
- Reduction of intersection conflict points for both vehicles and pedestrians.
- General increase in safety performance.
Design Considerations

The distance between the MUT crossovers is central to the proper functioning of the intersection. The AASHTO recommended spacing from the main intersection to the median opening is 400 to 600 ft.; although this distance may be modified per guidance in the FHWA Median U-Turn Informational Guide. Longer distances are preferred for weaving, but as the distance between the intersection and median opening increases, the delay for the minor road left-turn and through vehicles increases. See Figures E-28 through E-30 for spacing considerations.

![Figure E-28. Spacing Consideration for a Major Street Left Turn Movement](source: FHWA MUT Informational Guide - Exhibit 7-15)
Figure E-29. Spacing Consideration for Minor Street Left Turn Movement

Figure E-30. Spacing consideration for a Right Turn
The MUT median width is a function of the design vehicle (typically a WB-67 in Texas) and the preferred alignment of the design vehicle as it turns into the opposing traffic (see Figure E-31); the “m” in Figure E-31 is the minimum median width measured in feet. For instances where the U-turn can’t be completed using the existing pavement, a loon may be used to provide additional space (see Figure E-32 as an example application for a longer weaving distance).

<table>
<thead>
<tr>
<th>Type of Maneuver</th>
<th>U.S. Customary</th>
<th>M—Minimum Width of Median (m) for Design Vehicle</th>
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<tr>
<td>Inner Lane to Inner Lane</td>
<td></td>
<td></td>
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<tr>
<td>Inner Lane to Outer Lane</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner Lane to Shoulder</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length of Design Vehicle (ft)</td>
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<td>50</td>
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<tr>
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<tr>
<td>WB-67</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source: FHWA MUT Informational Guide - Exhibit 7-12

Figure E-31. AASHTO Minimum Median Widths for U-Turn Crossovers

Source: FHWA MUT Informational Guide - Exhibit 7-13

Figure E-32. Loon Design Serving a Design Vehicle
Location Considerations

Providing a stop controlled median U-turn location before the main intersection would improve land access between the main intersection and the crossover, route flow, and driver assumptions. However, U-turn movements may conflict with right turn movements. Figure E-33 shows an example of median U-turn locations. Designers should also ensure good intersection sight distances at MUT crossovers by making sure slopes and plantings in the median are cut back beyond the lines of sight.

![Stop Controlled Median U-Turn](image)

*Figure E-33. Stop Controlled Median U-Turn*

Pedestrian and Bicyclist Considerations

MUTs have benefits for pedestrians which include a reduction in the number of conflict points and benefits due to two-phase signalization. Single and two-stage pedestrian crossings are shown in Figure E-34.
Bicyclists have three options for navigating a MUT intersection when making a left turn:

1. Bicyclists making a two-stage left turn (preferred option): Minor street bicyclists approach the intersection on the right and follow the vehicle signal indications. When receiving the green indication, the bicyclists proceed across the intersection and stop in a bicycle turn queue box. When the major street receives a green indication, bicyclists proceed along the major street.

2. Bicyclists follow pedestrian crossing rules: Bicyclists approach the intersection and exit the street to the right and follow the pedestrian (“walk”/”don’t walk”) indications.

3. Bicyclists following vehicle rules: Bicyclists approach the intersection on the right and follow the vehicle signal indications. This option is legally permissible, but undesirable.

Ultimately, a thorough site assessment, an assessment of anticipated bicycle and pedestrian volumes, and an assessment of projected origins and destinations for pedestrians and bicyclists should be conducted to determine the preferred method of movement through the MUT.

Figure E-35 shows the three bicyclist options:
Access Management

From an access management standpoint, possible advantages of a MUT include the elimination of left turns from driveways, and the consolidation of access to U-turn crossover intersections. Possible disadvantages include having to pass twice through intersections, and restricted access between main crossing and U-turn intersections. Reference the FHWA Alternative Intersections/Interchanges Informational Report (AIIR) for additional guidance with respect to access management.
Section 5 — Restricted Crossing U-Turn Intersection (RCUT)

Overview

The RCUT, also known as a superstreet intersection, is an at-grade intersection with directional medians such that the minor road traffic must turn right and make a U-turn back to cross or make a left-turn maneuver. The benefits of the RCUT include a significant reduction in intersection conflict points, and a reduction in crash rates and crash severity. TxDOT has adopted the FHWA Restricted Crossing U-Turn Informational Guide as the primary source for RCUT Guidelines. The information contained in this Appendix is considered a companion guide and is intended to document TxDOT’s suggested approach to RCUT design.

See Figures E-36 through E-38 for examples of RCUTs with signals, without signals, and with merges. For RCUT 3 leg approaches, reference Chapter 7 of the FHWA Restricted Crossing U-Turn Informational Guide.

Figure E-36. Example of RCUT Intersection with Signals
**Design Considerations**

The spacing of the RCUT crossovers from the intersection is central to the proper functioning of the intersection. The typical location for the median opening at an RCUT intersection with stop sign or signal control is 400 to 800 ft. downstream of the minor road.
The median width is a function of the design vehicle (typically a WB-67 in Texas) and the preferred alignment of the design vehicle as it turns into the opposing traffic (see Figure E-31). For instances where the U-turn can’t be completed using the existing pavement, a loon type configuration with back-to-back storage bays may be used to provide additional space (see Figure E-39). See Figure E-40 for spacing considerations for a minor street through or left movement. Designers should also ensure good intersection sight distances at RCUT crossovers by making sure slopes and plantings in the median are cut back beyond the lines of sight.

Source: FHWA RCUT Informational Guide - Exhibit 7-19

Figure E-39. RCUT Intersection with Back-to-Back Two-Lane Crossover Storage Bays
The angle of the side roads to the main lanes of an RCUT intersection can have an impact on traffic operations and the conversion from a conventional intersection to an RCUT intersection. RCUT intersections at an acute angle (less than 90 degrees) generate left turn movements more effectively with the major street as opposed to an obtuse angle (greater than 90 degrees). Figure E-41 demonstrates this concept.

Source: FHWA RCUT Informational Guide - Exhibit 7-22

Figure E-40. Spacing Consideration for a Minor Street Through or Left Movement

Intersection Angle
Pedestrian and Bicyclist Considerations

RCUTs have benefits for pedestrians, which include a reduction in the number of conflict points and a reduction in cycle lengths (when signalized). In general, an RCUT is better suited for relatively low pedestrian activity or, at least, relatively low volumes of pedestrians crossing the major street. The primary pedestrian crossing pattern is shown in Figure E-42. The *FHWA Restricted Crossing U-Turn Informational Guide* provides guidance for additional pedestrian crossing alternatives.
Bicyclists on the major roadway travel through the RCUT the same way they travel through a conventional intersection. The options available for bicyclists approaching on the minor street are:

- Bicyclists follow pedestrian crossing rules (preferred option): Bicyclists approach the intersection and exit the street to the right and follow the pedestrian (“walk” / “don’t walk”) indications.
- If no crosswalk is available, a potential option is for the bicyclist to pass through/across the channelizing island.
- Bicyclists following vehicle rules: Bicyclists approach the intersection on the right and follow the vehicle signal indications. This option is legally permissible, but undesirable.

Ultimately, a thorough site assessment, an assessment of anticipated bicycle and pedestrian volumes, and an assessment of projected origins and destinations for pedestrians and bicyclists should be conducted to determine the preferred method of movement through the RCUT.

Figure E-43 depicts the three options:
Access Management

From an access management standpoint, some of the possible advantages of an RCUT include the following:

- Provides multiple side street locations along the RCUT corridor.
- Allows flexibility for crossover locations to accommodate adjacent driveways and side streets.
- Provides significant progression benefits along the corridor, which can allow for speed control using the signals.

Some of the possible disadvantages include not allowing driveways or side streets near entrances to U-turn crossovers and not having driveways with direct left turns. In general, avoid access points for 100 ft. on either side of the entrance to a U-turn crossover (see Figure E-44). Reference the FHWA Alternative Intersections/Interchanges Informational Report (AIIR) for additional guidance with respect to access management.
Generally avoid access points for 100 feet on either side of the entrance to a U-turn crossover.

Source: FHWA RCUT Informational Guide - Exhibit 7-14

Figure E-44. Area Near U-turn Crossover Where Access Point Should Be Avoided
Section 6 — Displaced Left Turn Intersection (DLT)

Overview

The DLT, also known as a continuous flow intersection (CFI) is an intersection that relocates one or more left turn movements on an approach to the other side of the opposing traffic flow. This allows left-turn movements to proceed simultaneously with the through movements and eliminates the left-turn phase for this approach. TxDOT has adopted the *FHWA Displaced Left Turn Intersection Informational Guide* as the primary source for DLT Guidelines. The information contained in this Appendix is considered a companion guide and is intended to document TxDOT’s suggested approach to DLT design. See Figure E-45 for a typical four-legged DLT with displaced lefts on a major street. (For a DLT 3 leg intersection reference Chapter 7 of the FHWA DLT Informational Guide)

![Diagram of a four-legged DLT with displaced lefts on a major street](source)

*Figure E-45. Four-legged DLT with Displaced Lefts on a Major Street*

Benefits of a DLT include a reduction in the number of signal phases with a resultant increase in vehicle throughput. See Figure E-46 for an example of a DLT Intersection with displaced left turns on all approaches.
Figure E-46. DLT Intersection with Displaced Left Turns on All Approaches

Design Considerations

The distance between the main intersection and the crossovers should generally range from 300 to 500 ft. Shorter spacing may result in queue spillback and reduce the ability to clear queues through a single signal cycle phase. Longer spacing may result in greater difficulty in coordinating signal operations. See Figure E-47 for a depiction of typical intersection spacing.
At the main intersection, the primary objective is to provide appropriate turning paths (typically a WB-67 in Texas) for the displaced left turns, considering the interaction with sidewalks. The vehicle paths for the displaced left turns through the main intersection will delineate the curb lines and stop bar locations and determine the width of the overall intersection as shown in Figure E-48.

Source: FHWA DLT Informational Guide - Exhibit 7-18

Figure E-47. DLT Typical Intersection Spacing
For the crossover intersection, the main objective is to provide a smooth alignment for the traffic and not introduce back-to-back reverse curves along the travel paths. The goal is to align the left turns at the stop bar with the receiving lanes to reflect desirable vehicle path alignment to minimize path overlap as shown in Figure E-49.

Additionally, there are two ways to accommodate the geometry where the right-turn bypass lane joins the cross road through lanes: 1) Provide an additional lane with a downstream merge, as
shown in Figure E-50. 2) Signalize the movement and operate it as part of the crossover signal, as shown in Figure E-51.

![Add Lane with a Downstream Lane Merge](source)

**Figure E-50. Add Lane with a Downstream Lane Merge**

![Signalized Right Turn](source)

**Figure E-51. Signalized Right Turn**
Sight Distance

Provide appropriate sight distance and lighting for approaching motorists to see activity at the crosswalk, as well as sight distance for pedestrians to see oncoming traffic. Stopping sight distance for the approaching motorists and sight distances for the pedestrians approaching the potential oncoming automobiles should be clear of obstructions and provide sufficient visibility for various users.

Pedestrian and Bicyclist Considerations

Crosswalks for DLTs are at the same locations that they would be for a conventional intersection. The major street crossing could be made in one or two stages, where the median can be used to provide a refuge for a two-stage crossing. Figure E-52 shows a typical pedestrian crossing with refuge islands.

![Refuge Islands Between Left-Turn and Through Lanes](source)

Figure E-52. Refuge Islands Between Left-Turn and Through Lanes

There are various options for bicyclists to use at a DLT; Figures E-53 and E-54 show options for bicycle through movements on and off street, respectively, and Figures E-55 and E-56 show options for bicycle left-turn movements on and off street, respectively. Ultimately, a thorough site assessment, an assessment of anticipated bicycle and pedestrian volumes, and an assessment of projected origins and destinations for pedestrians and bicyclists should be conducted to determine the preferred method of movement through the DLT.
Section 6 — Displaced Left Turn Intersection (DLT)

Figure E-53. Accommodating On-Street Bicycles Through a DLT Intersection

Figure E-54. Accommodating Off-Street Bicycles Through a DLT Intersection
Figure E-55. Accommodating On-Street Left-Turning Bicycles with a Bicycle Box Through a DLT Intersection

Source: FHWA DLT Informational Guide - Exhibit 3-12

Figure E-56. Accommodating Off-Street Left-Turning Bicycles Through a DLT Intersection

Source: FHWA DLT Informational Guide - Exhibit 3-13
Access Management

Maintaining or providing access to homes and businesses near a DLT intersection can be accomplished using frontage roads; however, the following operational impacts may result: Weaving movements in and out of driveways or U-turns (at the main or adjacent intersections) may be in conflict. DLT intersection implementation typically restricts access to parcels situated in the quadrants of the main intersection. Access to these parcels can be accommodated via right-in/right out from the channelized right-turn lanes. Reference the NCHRP Report 420 "Impacts of Access Management Techniques" for additional guidance for access management for DLT intersections.
Section 7 — References


