Roadway Design Manual

Texas
Department
of Transportation

Revised December 2013

Manual Notice 2013-1

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Manual: *Roadway Design Manual*

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Purpose

This revision is intended to update the *Roadway Design Manual*, specifically Chapter 2, Basic Design Criteria, based on 2011 American Association of State Highway and Transportation Officials (AASHTO) *Geometric Design of Highways and Streets* revisions.

Contents

The following changes have been made:

Chapter 1, Preface

- Added non-discrimination statement.

Chapter 1, Section 2, Design Exceptions, Design Waivers and Design Variances

- Subsection "New Location and Reconstruction Projects (4R)" - Added "Lateral offset to Obstructions" to list of controlling criteria that require a design exception.

Chapter 2, Basic Design Criteria

- General renumbering of Tables and Figures throughout the chapter,
- Updated English and Metric tables to current AASHTO values, and
- Metric Versions are no longer shown but may be viewed in PDF format by accessing link.

Chapter 2, Section 3, Sight Distance

- Subsection "Decision Sight Distance" - paragraph added at the end of subsection regarding a reduced decision zone,
- Subsection "Passing Sight Distance" - added reference to Super 2 Highways for further discussion, and
- Subsection "Intersection Sight Distance" - the second bullet describing the types of intersection controls has been rewritten.

Chapter 2, Section 4, Horizontal Alignment. This section has been completely reorganized for better flow and revisions are as follows:

- Table 2-3, "Horizontal Curvature of High-Speed Highways and Connecting Roadways with Superelevation", updated to current AASHTO values.
- Table 2-4, "Horizontal Curvature of Highways without Superelevation", updated to current AASHTO values. Values for 6% superelevation were added to table.
- Table 2-5, "Minimum Radii and Superelevation for Low-Speed Urban Streets", updated to current AASHTO values.
- Table 2-6, "Minimum Radii for Design Superelevation Rates, Design Speeds", and e_{max} = 6%, updated to current AASHTO values.
- Table 2-7, "Minimum Radii for Design Superelevation Rates", Design Speeds, and e_{max} = 8%, updated to current AASHTO values.
- Subsection "Superelevation Transition Length" - The first two paragraphs and Table 2-8 were relocated from the discussion on Superelevation in the current version of the manual. The last paragraph of this subsection and Table 2-9 "Multilane Adjustment Factor" were added.
- Subsection "Superelevation Transition Placement" -This is a new section and provides updated guidance on placement of the transition, and Table 2-10, "Portion of Superelevation Transition Located on the Tangent", where were added from the current AASHTO "Green Book".
- Subsection "Superelevation Transition Type" - This section is new and references the AASHTO "Green Book" for additional methods.
- Subsection "Sight Distance on Horizontal Curves" - The last paragraph in the section was added regarding methods to mitigate barrier obstructions.

Chapter 2, Section 5, Vertical Alignment

- Subsection "Crest Vertical Curves" - A sentence was added to the 3rd paragraph regarding successive vertical curves.
- Subsection "Combination of Vertical and Horizontal Alignment" - The last bullet was modified to include reference to Super 2 Highways.

Chapter 2, Section 6, Cross Sectional Elements

- The term "horizontal clearance" was replaced with "clear zone" throughout this section.
- Updated this section to current Americans with Disabilities Act Public Accessibility Guideline for Pedestrian Facilities in the Public Right of Way (PROWAG) and Texas Accessibility Standards (TAS).
- Subsection "Overview"
  - Added "Lateral Offset to Obstructions".
  - Changed "Horizontal Clearances to Obstructions" to "Clear Zone".
  - Moved "Slopes and Ditches" section down after "Roadside Design".

- Subsection "Pavement Cross Slope" - The last paragraph was modified to delete reference to design exception.
- Subsection "Lane Widths" - Added last paragraph discussing Bicycle Accommodations.
- Subsection "Shoulder Widths" - Added 2nd paragraph regarding bicycle accommodations and offset to barriers.
- Subsection "Sidewalks and Pedestrian Elements "- 1st paragraph was modified and factors when sidewalks should be included were added.
- Subsection "Sidewalk Location" - The buffer space between back of curb and sidewalk was modified. Additional discussion was added at the end of paragraph.
- Subsection "Sidewalk Width" - minor wording modifications to the two paragraphs.
- Subsection "Street Crossing" - Dimensions for refuge islands and cut throughs were modified. Last line of paragraph was added.
- Subsection "Curb Ramps and Landings" - modified 1st bullet to include reconstruction and rehabilitation. Added preferred width of curb ramps.
- Subsection "Cross Slope" - added sentence to end of paragraph regarding the running slope of sidewalks across driveways.
- Subsection "Street Furniture" - added a sentence to last paragraph discussing pedestrian push buttons.
- Subsection "Slopes and Ditches" - relocated discussion. Deleted Table 2-10.
- Subsection "Lateral Offset to Obstruction" - New section added.
- Subsection "Clear Zone" - Changed section title from "Horizontal Clearances to Obstructions" to "Clear Zone" and discussion on definition.
- Table 2-12 - The clear zone width changed from 1.5 ft to 4 ft minimum, and 3 ft to 6 ft desirable for "Urban, curbed, < 45 mph".

Chapter 2, Section 7, Drainage Facility Placement

- The term "horizontal clearance" was replaced with "clear zone" throughout this section.
- Subsection "Overview" - added "Side Ditches".
- Subsection "Design Treatment of Cross Drainage Culvert Ends" - Bullets removed from first paragraph. Discussed in subsequent sections.
- Subsection "Small Pipe Culverts" - combined paragraphs 3 and 4.
- Subsection "Intermediate Size Single Box Culverts and (Single and Multiple) Pipe Culverts" - 2nd recommended safety treatment option not applicable to bridge class culverts.
- Subsection "Multiple Box Culverts and Large single Pipes or Boxes": - 2nd recommended safety treatment option not applicable to bridge class culverts.

- Subsection "Bridge Class Drainage Culverts" - added recommended treatment options to 2nd paragraph. Modified Table 2-14 and its title.
- Figure 2-10, "Use of Guardrail at Bridge Class Culverts", depth of cover changed to ">9" to < 36".
- Figure 2-11, "Use of Sloping Pipe Ends Without Cross Pipes", replaced the term Grates with Cross Pipes.
- Figure 2-12, "Use of Sloping Pipe Ends with Cross Pipes", replaced the term Grates with Cross Pipes.
- Subsection "Side Ditches" - added wording in the last paragraph regarding rock filter dam.

Instructions

This revision will be distributed online only.

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 superelevation criteria, this manual, and all revisions, will be effective for all projects beginning with the December 2014 letting. Project development using this manual and its revisions prior to that date is at the option of the district.

Contact

For general comments and suggestions for future revisions of this manual, contact the Design Division, Roadway Design Section.

Copyright Notice

Archives

Past manual notices are available in a pdf archive.

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Chapter 5 — Non-Freeway Resurfacing or Restoration Projects (2R)

Chapter 6 — Special Facilities

Chapter 7 — Miscellaneous Design Elements

Preface

The *Roadway Design Manual* was developed by the Texas Department of Transportation 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

Application of Design Guidelines

The criteria contained in this *Roadway Design Manual* (manual) are applicable to all classes of highways from freeways to two-lane roads. This manual 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 manual is formatted to follow the traditional resurfacing, restoration, rehabilitation, and reconstruction (the four R's) of highway construction. The individual sections are briefly described in the following paragraphs.

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 manual is broken into roadway classifications such as urban streets, suburban roadways, two-lane highways, multilane 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 nonfreeway 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, parking, emergency median openings, and minimum turning designs. These individual design elements can be selected as needed and incorporated into appropriate project designs.

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.

External Reference Documents

It is recommended that the following publications, in their current editions, be available for reference in conjunction with this manual. 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*, American Association of State Highway and Transportation Officials (AASHTO).
- *Highway Capacity Manual*, Transportation Research Board (TRB).
- *Guide for the Development of Bicycle Facilities*, American Association of State Highway and Transportation Officials (AASHTO).
- *Guide for the Design of High Occupancy Vehicle Facilities*, American Association of State Highway and Transportation Officials (AASHTO)

The American Association of State Highway and Transportation Officials (AASHTO) has established various policies, standards, and guides relating to transportation design practices. These documents are approved references to be used in conjunction with this manual. However, the instructions given in this manual will take precedence over AASHTO documents unless specifically noted otherwise.

Section 2 — Design Exceptions, Design Waivers and Design Variances

Overview

This subsection discusses the following topics:

- design exceptions
- design waivers
- design variances

Design Exceptions

A design exception is required whenever the criteria for certain 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. The determination of whether a design exception exists rests with the district, unless the project is subject to federal oversight or review. A design exception is not required when values exceed the guidelines for the controlling criteria.

Design exceptions for plans, specifications and estimates, designated federal oversight under the current Federal Oversight Agreement must be reviewed and approved by the FHWA. Design exceptions for all schematics on the NHS with the exception of preventive maintenance, freeway safety and 3R type projects must be reviewed and approved by the FHWA.

Design exceptions for all projects on the interstate system must also be reviewed and approved by the FHWA.

Design exceptions involving the structural capacity or bridge width shall be sent to the Bridge Division for their review and approval.

Final approval of a roadway design exception must be signed by the district engineer and this signature authority cannot be delegated. 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 reviews of any three of the four member committee would constitute a quorum for recommending signature action.

The complete documentation for a roadway exception should be retained permanently in the district project files and a copy furnished to the Design Division. 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 that will require a design exception.

- Design Speed
- Lane Width
- Shoulder Width
- Bridge Width (see Bridge Project Development Manual)
- Structural Capacity (see Bridge Project Development Manual)
- Horizontal Alignment
- Vertical Alignment
- Grades
- Stopping Sight Distance
- Cross Slope
- Superelevation
- Vertical Clearance
- Lateral offset to obstructions

Resurfacing, Restoration or Rehabilitation (3R) Projects. The list below gives the controlling criteria 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 Alignment (high volume roadways)
- Vertical Alignment (high volume roadways)
- Superelevation (high volume roadways)
- Grades (high volume roadways)

- Lane Width
- Shoulder Width
- Bridge Width (see Bridge Project Development Manual)
- Structural Capacity (see Bridge Project Development Manual)

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.

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
- Structural Capacity (see Bridge Project Development Manual)
- Horizontal Alignment
- Vertical Alignment
- Grades
- Cross Slope
- Superelevation
- Minimum Structure Width, Face to Face of Rail: 24 ft [7.2 m].

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. 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 must meet the required design criteria for the appropriate roadway classification including exception or waiver requirements.

Bicycle Facilities. Design exceptions are necessary when the minimum requirements given in the AASHTO *Guide for the Development of Bicycle Facilities* for on-street bicycle lanes and increased shared lane width cannot be met.

Design Waivers

When the criteria is not met in a noncontrolling category, a design exception is not required. However, variations from the criteria in these cases 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 files and a copy furnished to the Design Division.

The following project categories will have noncontrolling criteria that dictate a design waiver.

New Location and Reconstruction Projects (4R). The list below gives the noncontrolling criteria that will require a design waiver:

- Curb Parking Lane Width
- Speed Change (refuge) Lane Width
- Length of Speed Change Lanes
- Curb Offset
- Median Opening Width
- Horizontal Clearance (clear zone)
- Railroad Overpass Geometrics
- Guardrail Length (unless for access accommodation; see Appendix A, Metal Beam Guardrails).

Resurfacing, Restoration or Rehabilitation (3R) Projects. The list below gives the noncontrolling criteria that will require a design waiver. For 3R projects, low volume roadways are defined as current ADT of less than 1500.

- Design Speed (low volume roadways)
- Horizontal Alignment (low volume roadways)
- Vertical Alignment (low volume roadways)
- Superelevation (low volume roadways)
- Grades (low volume roadways)
- Deficient Bridge Rails (low volume roadways)
- Obstruction Clearance (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).

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

Special Facilities. Design waivers are not applicable to special facility projects including (1) off-system bridge replacement and rehabilitation projects, (2) off-system historically significant bridge projects, or (3) park road projects.

Design waivers are necessary when the minimum requirements given in the *AASHTO Guide for the Development of Bicycle Facilities* for separate bicycle paths cannot be met.

Design Variances

A design variance is required whenever the design guidelines specified in the *Americans with Disabilities Act Accessibility Guidelines (ADAAG)* and the Texas Accessibility Standards are not met. Design variances should be sent to the Design Division for forwarding to the Texas Department of Licensing and Regulation for approval. Refer to Sidewalks and Pedestrian Elements in Chapter 2 for additional discussion.

Section 3 — Schematic Layouts

Overview

The submission of schematic layouts should include the basic information necessary for the proper review and evaluation of the proposed improvement:

- General project information including project limits, design speed, and functional classification.
- The location of interchanges, mainlanes, grade separations, frontage roads, turnarounds, and ramps.
- Existing and proposed profiles and horizontal alignments of mainlanes, ramps, and crossroads at proposed interchanges or grade separations. Frontage road alignment data need not be shown on the schematic, however, it should be developed in sufficient detail to determine right of way needs.
- For freeways, the location and text of the proposed mainlane guide signs should be shown. Lane lines and/or arrows indicating the number of lanes should be shown.
- For freeway added capacity projects, a capacity analysis.
- An explanation of the sequence and methods of stage construction including initial and ultimate proposed treatment of crossovers and ramps.
- The tentative right of way limits.
- Bridges and bridge class culverts should be shown.
- The geometrics (pavement cross slope, superelevation, lane and shoulder widths, slope ratio for fills and cuts) of the typical sections of proposed highway mainlanes, ramps, frontage roads, and cross roads.
- Location of retaining walls and/or noise walls.
- The existing and proposed traffic volumes and, as applicable, turning movement volumes.
- If applicable, the existing and proposed control of access lines.
- The direction of traffic flow on all roadways.
- If applicable, location and width of median openings.
- The geometrics of speed change and auxiliary lanes.
- Design speed.
- Existing roadways and structures to be closed or removed.

Section 4 — Additional Access to the Interstate System

Requirements

According to the Code of Federal Regulations, 23 CFR 630, proposals for new or revised access points to the existing interstate system should meet the following requirements:

- The existing interchanges and/or local roads and streets in the corridor can neither provide the necessary access nor be improved to satisfactorily accommodate the design year traffic demands while at the same time providing the access intended by the proposal.
- All reasonable alternatives for design options, location and transportation system management type improvements (such as ramp metering, mass transit, and HOV facilities) have been assessed and provided for if currently justified, or provisions are included for accommodating such facilities if a future need is identified.
- The proposed access point does not have a significant adverse impact on the safety and operation of the interstate facility based on an analysis of current and future traffic. The operational analysis for existing conditions shall, particularly in urbanized areas, include an analysis of sections of interstate to and including at least the first adjacent existing or proposed interchange on either side. Crossroads and other roads and streets shall be included in the analysis to the extent necessary to assure their ability to collect and distribute traffic to and from the interchange with new or revised access points.
- The proposed access connects to a public road only and will provide for all traffic movements. Less than "full interchanges" for special purpose access for transit vehicles, for HOV's, or into park and ride lots may be considered on a case by case basis. The proposed access will be designed to meet or exceed current standards for federal aid projects on the interstate system.
- The proposal considers and is consistent with local and regional land use and transportation plans. Prior to final approval, all requests for new or revised access must be consistent with the metropolitan and/or statewide transportation plan, as appropriate, the applicable provisions of 23 CFR part 450 and the transportation conformity requirements of 40 CFR parts 51 and 93.
- In areas where the potential exists for future multiple interchange additions, all requests for new or revised access are supported by a comprehensive interstate network study with recommendations that address all proposed and desired access within the context of a long term plan.
- The request for a new or revised access generated by new or expanded development demonstrates appropriate coordination between the development and related or otherwise required transportation system improvements.
- The request for new or revised access contains information relative to the planning requirements and the status of the environmental processing of the proposal.

  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 includes routes incorporated into the interstate system under the provisions of 23 U.S.C. 139(a) or other legislation. Routes approved as a future part of the interstate system under 23 U.S.C. 139(b) represent a special case because they are not yet a part of the interstate system and the requirements contained herein do not apply. However, since the intention to add the route to the interstate system has been formalized by agreement, any proposed access points, regardless of funding, must be coordinated with the FHWA Division Office.
- These requirements are not applicable to toll roads incorporated into the interstate system, except for segments where federal funds have been expended, or where the toll road section has been added to the interstate system under the provisions of 23 U.S.C. 139(a).
- Each entrance or exit point, including "locked gate" access, to the mainlanes is considered to be an access point. For example, a diamond interchange configuration has four access points. Generally, revised access is considered to be a change in 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.
- 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 National Environmental Policy Act (NEPA) procedures are followed. The NEPA procedures will be accomplished as part of the normal project development process and as a condition of the access approval. This means the final approval of access cannot precede the completion of the NEPA process. To offer maximum flexibility, however, any proposed access points can be submitted in accordance with the delegation of authority for a determination of engineering and operational acceptability prior to completion of the NEPA process. In this manner, the state highway agency can determine if a proposal is acceptable for inclusion as an alternative in the environmental process. These requirements in no way alter the current implementing procedures as contained in 23 CFR part 771.
- Although the justification and documentation procedures can be applied to access requests for non-interstate freeways or other access controlled highways, they are not required. However, applicable federal rules and regulations, including NEPA procedures, must be followed.

  The request should contain sufficient information to independently evaluate the proposal and ensure that all pertinent factors and alternatives have been appropriately considered. The extent and format of the required documentation and justification should be consistent with the complexity and expected impact of the proposal. No specific documentation format or content is prescribed. The Design Division can provide assistance with documentation and examples of proposals. The final documentation for these requests should be sent to the Design Division for coordination with the FHWA Division Office.

Section 5 — Preliminary Design Submissions

Submissions

The preliminary submission should clearly establish the design criteria or guidelines under which the project is being developed. The following table outlines preliminary design items that should be submitted.

Preliminary Design Submission

Item	Submission
Design Summary Report ◆ Form 1002 with applicable design speed and design criteria ◆ Typical section	As soon after project authorization as practical, submit to DES, Field Coordination.
Pavement design	With copy of typical sections to Pavement Design Section, DES, as soon after project authorization as practical.
Schematic layout	Submit to DES, Field Coordination prior to initiating detailed plan preparation.
Exhibit layouts for work on railroad rights of way for railroad agreements	Refer to *Traffic Operations Manual,* Railroad Operations Volume.
Bridge layouts	Submit in accordance with the Bridge Project Development *Manual.*
Hike/Bike facility schematic	Submit to DES, Field Coordination prior to initiating detailed plan preparation.

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.

Different areas can be expected to have different maintenance considerations. 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:

- 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 variations (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.).

To the extent practical, utilization of desirable design criteria recommended herein regarding maximum roadway sideslope 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

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
- performing weaving analysis and subsequent determination of weaving section lengths

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

Traffic Speed

Traffic speed is influenced by volume, capacity, design, weather, traffic control devices, posted speed limit, and individual driver preference. For design purposes, the following definitions apply:

- Low-speed is 45 mph [70 km/h] and below
- 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. It is important to design facilities with all elements in balance, consistent with an appropriate design speed. Design elements such as sight distance, vertical and horizontal alignment, lane and shoulder widths, roadside clearances, superelevation, etc., are influenced by design speed.

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. Whenever mountainous conditions are encountered, refer to AASHTO's *A Policy on Geometric Design for Highways and Streets*.

Posted Speed. Posted speed refers to the maximum speed limit posted on a section of highway. TxDOT's *Procedure for Establishing Speed Zones* 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.

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`."

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"
- "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 a 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

			Stopping sight distance	
Design Speed (mph)	**Brake reaction distance (ft)**	**Braking distance on level (ft)**	**Calculated (ft)**	**Design (ft)**
15	55.1	21.6	76.7	80
20	73.5	38.4	111.9	115
25	91.9	60.0	151.9	155
30	110.3	86.4	196.7	200
35	128.6	117.6	246.2	250
40	147.0	153.6	300.6	305
45	165.4	194.4	359.8	360
50	183.8	240.0	423.8	425
55	202.1	290.3	492.4	495
60	220.5	345.5	566.0	570
65	238.9	405.5	644.4	645
70	257.3	470.3	727.6	730
75	275.6	539.9	815.5	820
80	294.0	614.3	908.3	910
Note: brake reaction distance predicated on a time of 2.5s; deceleration rate 11.2 ft/sec²				

NOTE: Online users can view the metric version of this table in PDF format.

Decision Sight Distance

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

Table 2-2: Recommended Decision Sight Distance Values

Decision sight distance (ft) Avoidance maneuver					
Design speed (mph)	**A**	**B**	**C**	**D**	**E**
30	220	490	450	535	620
35	275	590	525	625	720
40	330	690	600	715	825
45	395	800	675	800	930
50	465	910	750	890	1030
55	535	1030	865	980	1135
60	610	1150	990	1125	1280
65	695	1275	1050	1220	1365
70	780	1410	1105	1275	1445
75	875	1545	1180	1365	1545
80	970	1685	1260	1455	1650
Avoidance Maneuver A: Stop on rural road – t = 3.0s					
Avoidance Maneuver B: Stop on urban road – t = 9.1s					
Avoidance Maneuver C: Speed/path/direction change on rural road – t varies between 10.2 and 11.2s					
Avoidance Maneuver D: Speed/path/direction change on suburban road – t varies between 12.1 and 12.9s					
Avoidance Maneuver E: Speed/path/direction change on urban road – t varies between 14.0 and 14.5s					

NOTE: Online users can view the metric version of this table in PDF format.

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
- 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 reduce 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 should be clearly visible at all times.
- Lane markings and signs should be clearly visible and understandable from a desired distance.
- Intersections should be free from the sudden appearance of potential conflicts.
- Intersections should be evaluated for the effects of barriers, rails, and retaining walls on sight distance.

For selecting appropriate 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.

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 curvature is shown in Table 2-3 and "`Table 2-4: Horizontal Curvature of Highways without Superelevation`[1]."

Table 2-3: Horizontal Curvature of High-Speed Highways and Connecting Roadways with Superelevation

Design Speed (mph)	Usual Min.[1,2] Radius of Curve (ft)	Absolute Min.[1,3] Radius of Curve (ft)
[based on emax = 6%]		
45	810	643
50	1050	833
55	1635	1060
60	2195	1330
65	2740	1660
70	3390	2040
75	3750	2500
80	4575	3050
[based on emax = 8%]		
45	740	587
50	955	758
55	1480	960
60	1980	1200
65	2445	1480
70	3005	1810
75	3315	2210
80	4005	2670

[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 accident history indicates flattening curvature.

[3] Absolute minimum values should be used only where unusual design circumstances dictate.

NOTE: `Online users can view the metric version of this table in PDF format.`

Table 2-4: Horizontal Curvature of Highways without Superelevation

Design Speed (mph)	6% Min. Radius (ft)[1]	8% Min. Radius (ft)[1]
15	868	932
20	1580	1640
25	2290	2370
30	3130	3240
35	4100	4260
40	5230	5410
45	6480	6710
50	7870	8150
55	9410	9720
60	11100	11500
65	12600	12900
70	14100	14500
75	15700	16100
80	17400	17800
[1] Normal crown (2%) maintained		

NOTE: `Online users can view the metric version of this table in PDF format.`

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:

$e + f = V^2/15R$ **(US Customary)**

Where:

e = superelevation rate, in decimal format

f = side friction factor

V = vehicle speed, mph

R = curve radius, feet

NOTE: `Online users can view the metric version of this equation in PDF format.`

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
- 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
- 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

e(%)	V = 15 mph R (ft)	V = 20 mph R (ft)	V = 25 mph R (ft)	V = 30 mph R (ft)	V = 35 mph R (ft)	V = 40 mph R (ft)	V = 45 mph R (ft)
-4.0	54	116	219	375	583	889	1227
-3.0	52	111	208	353	544	821	1125
-2.8	51	110	206	349	537	808	1107
-2.6	51	109	204	345	530	796	1089
-2.4	51	108	202	341	524	784	1071
-2.2	50	108	200	337	517	773	1055
-2.0	50	107	198	333	510	762	1039
-1.5	49	105	194	324	495	736	1000
0	47	99	181	300	454	667	900
1.5	45	94	170	279	419	610	818
2.0	44	92	167	273	408	593	794
2.2	44	91	165	270	404	586	785
2.4	44	91	164	268	400	580	776
2.6	43	90	163	265	396	573	767
2.8	43	89	161	263	393	567	758
3.0	43	89	160	261	389	561	750
3.2	43	88	159	259	385	556	742
3.4	42	88	158	256	382	550	734
3.6	42	87	157	254	378	544	726
3.8	42	87	155	252	375	539	718
4.0	42	86	154	250	371	533	711

Notes:

1. Computed using Superelevation Distribution Method 2.
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%.

NOTE: Online users can view the metric version of this table in PDF format.

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: Minimum Radii for Design Superelevation Rates, Design Speeds, and emax = 6%

e (%)	15 mph R (ft)	20 mph R (ft)	25 mph R (ft)	30 mph R(ft)	35mph R (ft)	40 mph R (ft)	45 mph R (ft)	50 mph R (ft)	55 mph R (ft)	60 mph R (ft)	65mph R (ft)	70 mph R (ft)	75 mph R (ft)	80 mph R (ft)
2.0	614	1120	1630	2240	2950	3770	4680	5700	6820	8060	9130	10300	11500	12900
2.2	543	991	1450	2000	2630	3370	4190	5100	6110	7230	8200	9240	10400	11600
2.4	482	884	1300	1790	2360	3030	3770	4600	5520	6540	7430	8380	9420	10600
2.6	430	791	1170	1610	2130	2740	3420	4170	5020	5950	6770	7660	8620	9670
2.8	384	709	1050	1460	1930	2490	3110	3800	4580	5440	6200	7030	7930	8910
3.0	341	635	944	1320	1760	2270	2840	3480	4200	4990	5710	6490	7330	8260
3.2	300	566	850	1200	1600	2080	2600	3200	3860	4600	5280	6010	6810	7680
3.4	256	498	761	1080	1460	1900	2390	2940	3560	4250	4890	5580	6340	7180
3.6	209	422	673	972	1320	1740	2190	2710	3290	3940	4540	5210	5930	6720
3.8	176	358	583	864	1190	1590	2010	2490	3040	3650	4230	4860	5560	6320
4.0	151	309	511	766	1070	1440	1840	2300	2810	3390	3950	4550	5220	5950
4.2	131	270	452	684	960	1310	1680	2110	2590	3140	3680	4270	4910	5620
4.4	116	238	402	615	868	1190	1540	1940	2400	2920	3440	4010	4630	5320
4.6	102	212	360	555	788	1090	1410	1780	2210	2710	3220	3770	4380	5040
4.8	91	189	324	502	718	995	1300	1640	2050	2510	3000	3550	4140	4790
5.0	82	169	292	456	654	911	1190	1510	1890	2330	2800	3330	3910	4550
5.2	73	152	264	413	595	833	1090	1390	1750	2160	2610	3120	3690	4320
5.4	65	136	237	373	540	759	995	1280	1610	1990	2420	2910	3460	4090
5.6	58	121	212	335	487	687	903	1160	1470	1830	2230	2700	3230	3840
5.8	51	106	186	296	431	611	806	1040	1320	1650	2020	2460	2970	3560
6.0	39	81	144	231	340	485	643	833	1060	1330	1660	2040	2500	3050

NOTE: Online users can view the metric version of this table in PDF format.

Table 2-7: Minimum Radii for Design Superelevation Rates, Design Speeds, and emax = 8%

e (%)	V_d= 15mph R(ft)	V_d= 20mph R(ft)	V_d= 25mph R(ft)	V_d= 30mph R(ft)	V_d= 35mph R(ft)	V_d= 40mph R(ft)	V_d= 45mph R(ft)	V_d= 50mph R(ft)	V_d= 55mph R(ft)	V_d= 60mph R(ft)	V_d= 65mph R(ft)	V_d= 70mph R(ft)	V_d= 75mph R(ft)	V_d= 80mph R(ft)
2.0	676	1190	1720	2370	3120	3970	4930	5990	7150	8440	9510	10700	12000	13300
2.2	605	1070	1550	2130	2800	3570	4440	5400	6450	7620	8600	9660	10800	12000
2.4	546	959	1400	1930	2540	3240	4030	4910	5870	6930	7830	8810	9850	11000
2.6	496	872	1280	1760	2320	2960	3690	4490	5370	6350	7180	8090	9050	10100
2.8	453	796	1170	1610	2130	2720	3390	4130	4950	5850	6630	7470	8370	9340
3.0	415	730	1070	1480	1960	2510	3130	3820	4580	5420	6140	6930	7780	8700
3.2	382	672	985	1370	1820	2330	2900	3550	4250	5040	5720	6460	7260	8130
3.4	352	620	911	1270	1690	2170	2700	3300	3970	4700	5350	6050	6800	7620
3.6	324	572	845	1180	1570	2020	2520	3090	3710	4400	5010	5680	6400	7180
3.8	300	530	784	1100	1470	1890	2360	2890	3480	4140	4710	5350	6030	6780
4.0	277	490	729	1030	1370	1770	2220	2720	3270	3890	4450	5050	5710	6420
4.2	255	453	678	955	1280	1660	2080	2560	3080	3670	4200	4780	5410	6090
4.4	235	418	630	893	1200	1560	1960	2410	2910	3470	3980	4540	5140	5800
4.6	215	384	585	834	1130	1470	1850	2280	2750	3290	3770	4310	4890	5530
4.8	193	349	542	779	1060	1390	1750	2160	2610	3120	3590	4100	4670	5280
5.0	172	314	499	727	991	1310	1650	2040	2470	2960	3410	3910	4460	5050
5.2	154	284	457	676	929	1230	1560	1930	2350	2820	3250	3740	4260	4840
5.4	139	258	420	627	870	1160	1480	1830	2230	2680	3110	3570	4090	4640
5.6	126	236	387	582	813	1090	1390	1740	2120	2550	2970	3420	3920	4460
5.8	115	216	358	542	761	1030	1320	1650	2010	2430	2840	3280	3760	4290
6.0	105	199	332	506	713	965	1250	1560	1920	2320	2710	3150	3620	4140
6.2	97	184	308	472	669	909	1180	1480	1820	2210	2600	3020	3480	3990
6.4	89	170	287	442	628	857	1110	1400	1730	2110	2490	2910	3360	3850
6.6	82	157	267	413	590	808	1050	1330	1650	2010	2380	2790	3240	3720
6.8	76	146	248	386	553	761	990	1260	1560	1910	2280	2690	3120	3600
7.0	70	135	231	360	518	716	933	1190	1480	1820	2180	2580	3010	3480
7.2	64	125	214	336	485	672	878	1120	1400	1720	2070	2470	2900	3370
7.4	59	115	198	312	451	628	822	1060	1320	1630	1970	2350	2780	3250
7.6	54	105	182	287	417	583	765	980	1230	1530	1850	2230	2650	3120
7.8	48	94	164	261	380	533	701	901	1140	1410	1720	2090	2500	2970
8.0	38	76	134	214	314	444	587	758	960	1200	1480	1810	2210	2670

NOTE: Online users can view the metric version of this table in PDF format

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.

Table 2-8: Maximum Relative Gradient for Superelevation Transition

Design Speed (mph)	Maximum Relative Gradient%[1]	Equivalent Maximum Relative Slope
15	0.78	1:128
20	0.74	1:135
25	0.70	1:143
30	0.66	1:152
35	0.62	1:161
40	0.58	1:172
45	0.54	1:185
50	0.50	1:200
55	0.47	1:213
60	0.45	1:222
65	0.43	1:233
70	0.40	1:250
75	0.38	1:263
80	0.35	1:286

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

NOTE: Online users can view the metric version of this table in PDF format.

Transition length, L, for a multilane highway can be calculated using the following equation:

$L_{CT} = [(CS)(W)]/G$ **(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”).

NOTE: Online users can view the metric version of this equation in PDF format.

Example determinations of superelevation transition shown in Figure 2-1.

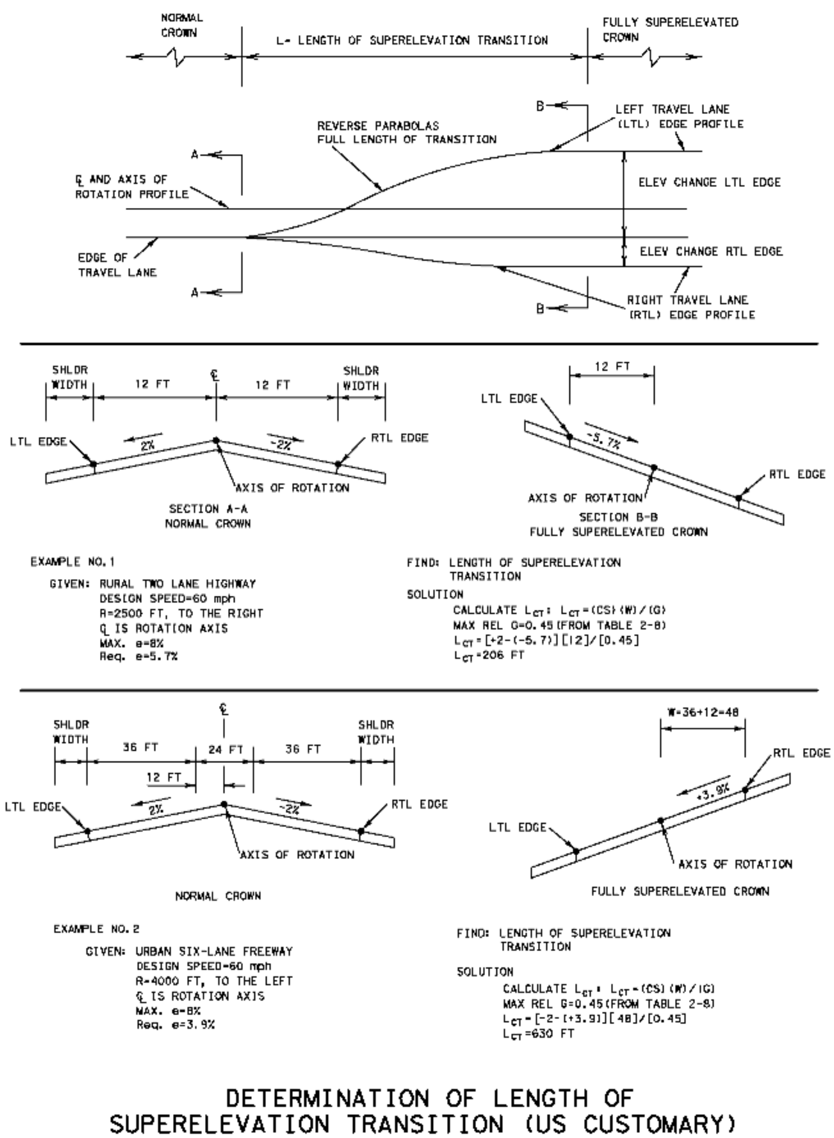


Figure 2-1. Determination of Length of Superelevation Transition. Click here to see a PDF of the image.

NOTE: Online users can view the metric version of this figure in PDF format.

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 (LCT) calculation is desirable, the length for multilane highways may become impractical for design purposes (e.g. drainage problems, avoiding bridges, accommodating merge/diverge condition). In such cases, an adjustment factor may be used to avoid excessive lengths such that the transition length formula becomes:

LCT = b[(CS)(W)]/G (US Customary and Metric)

where "b" is defined in Table 2-9

Table 2-9: Multilane Adjustment Factor1

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

[1] These adjustment factors are directly applicable to undivided streets and highways. For divided highways where the axis of rotation is not the edge of travel, see AASHTO's *A Policy on Geometric Design of Highways and Streets* discussion under "Axis of Rotation with a Median".

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 Tangent1

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

[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*, 2011, pg. 3-67).

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

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 2-1. 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 2-1 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)

NOTE: Online users can view the metric version of this equation in PDF format.

Figure 2-2 provides a graph illustrating the required offset where stopping sight distance is less than the length of curve (S<L).

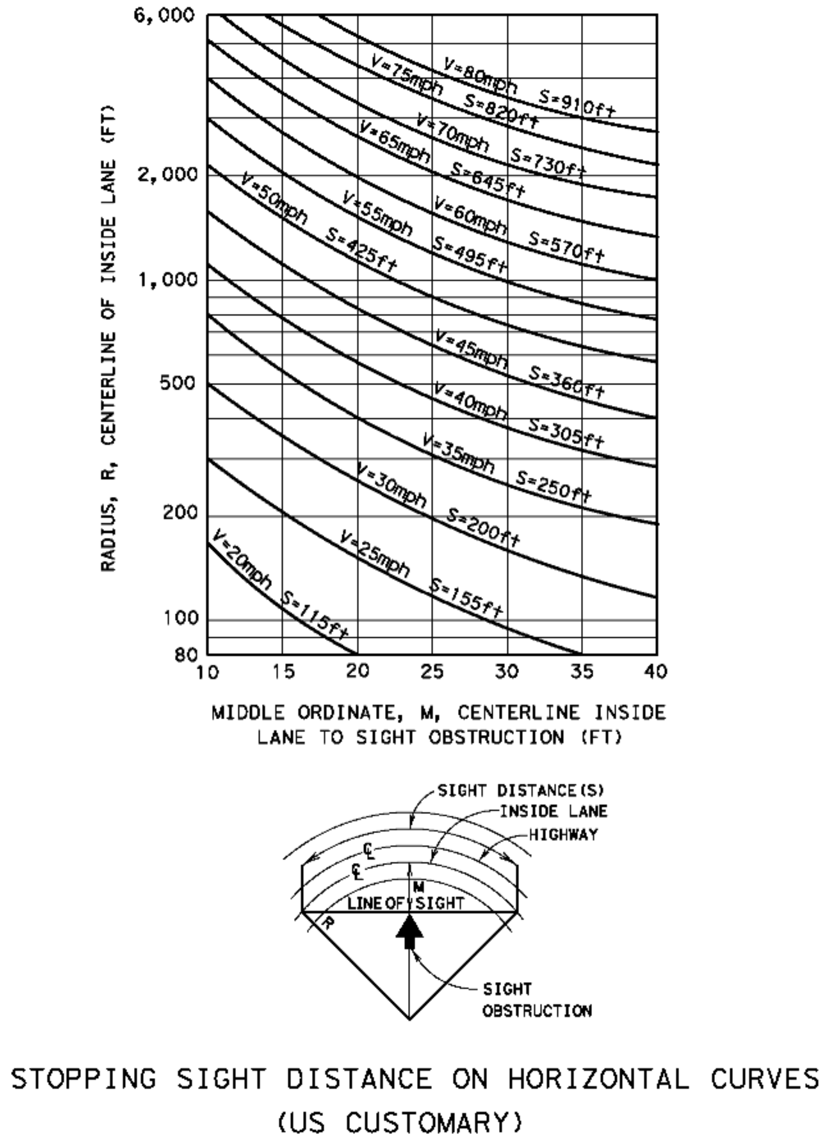


Figure 2-2. (US). Stopping Sight Distance on Horizontal Curves. Click here to see a PDF of the image.

NOTE: Online users can view the metric version of this figure in PDF format.

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) 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 2-3 shows the relationship of percent upgrade, length of grade, and truck speed reduction. Where critical length of grade is exceeded for two-lane highways, climbing lanes should be considered as discussed in the Transportation Research Board’s *Highway Capacity Manual*.

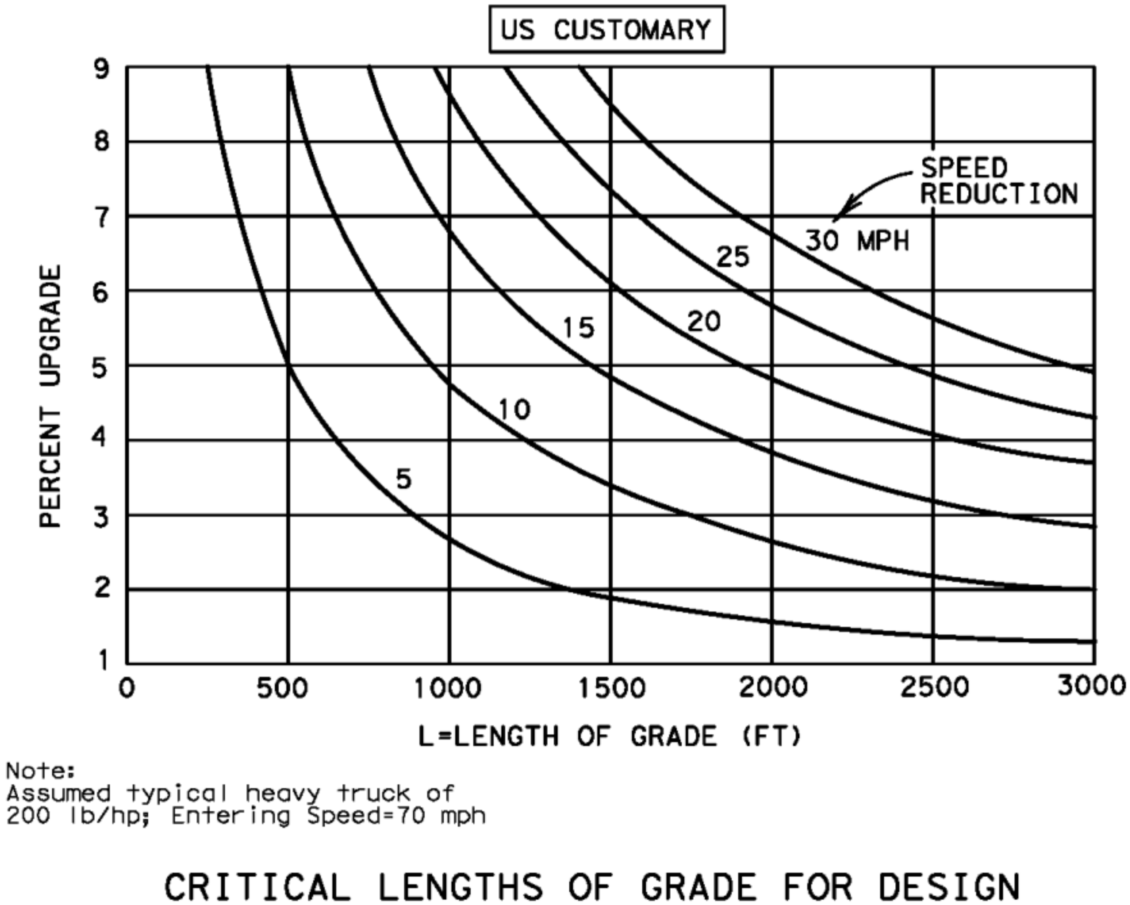


Figure 2-3. Critical Lengths of Grade for Design. Click here to see a PDF of the image.

NOTE: Online users can view the metric version of this figure in PDF format.

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

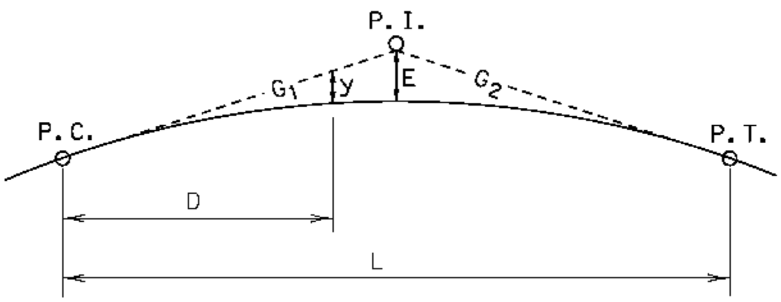
Functional Classification	Type of Terrain	15	20	25	30	35	40	45	50	55	60	65	70	75	80
Urban and Suburban:	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Local[1]	All	<15	<15	<15	<15	<15	<15	<15	--	--	--	--	--	--	--
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Collector	Level	9	9	9	9	9	9	8	7	7	6	--	--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

[1] 8% maximum in commercial areas on local streets, desirably less than 5%. Flatter gradients 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 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 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 2-4 is used in the highway profile design of vertical curves.



US CUSTOMARY EQUATION

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

2. $y = \frac{4 D^2 E}{L^2}$

   $y = \frac{D^2 A}{200L}$

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.T. to any point on curve, in feet

VERTICAL CURVE (US CUSTOMARY)

Figure 2-4. Vertical Curve. Click here to see a PDF of the image.

NOTE: Online users can view the metric version of this figure in PDF format.

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

L = 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)

Crest Vertical Curves. The minimum lengths of crest vertical curves for different values of A to provide the stopping sight distances for each design speed are shown in Figure 2-5. 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.

These minimum length of vertical curves (both crest and sag) are expressed as approximately three times the design speed in miles per hour (L_{min} = 3 V) or 0.6 times the design speed in kilometers per hour (L_{min} = 0.6 V). 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 ft [51 m] per percent change in grade which is plotted in Figures 2-5 and 2-6 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.

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. 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 2-6 with the solid lines representing the rounded K values. As with crest vertical curves, these lengths are minimum values based on design speed and longer curves are desired wherever practical. For sag vertical curves, drainage criteria and minimum curve lengths are established similarly to crest vertical curves.

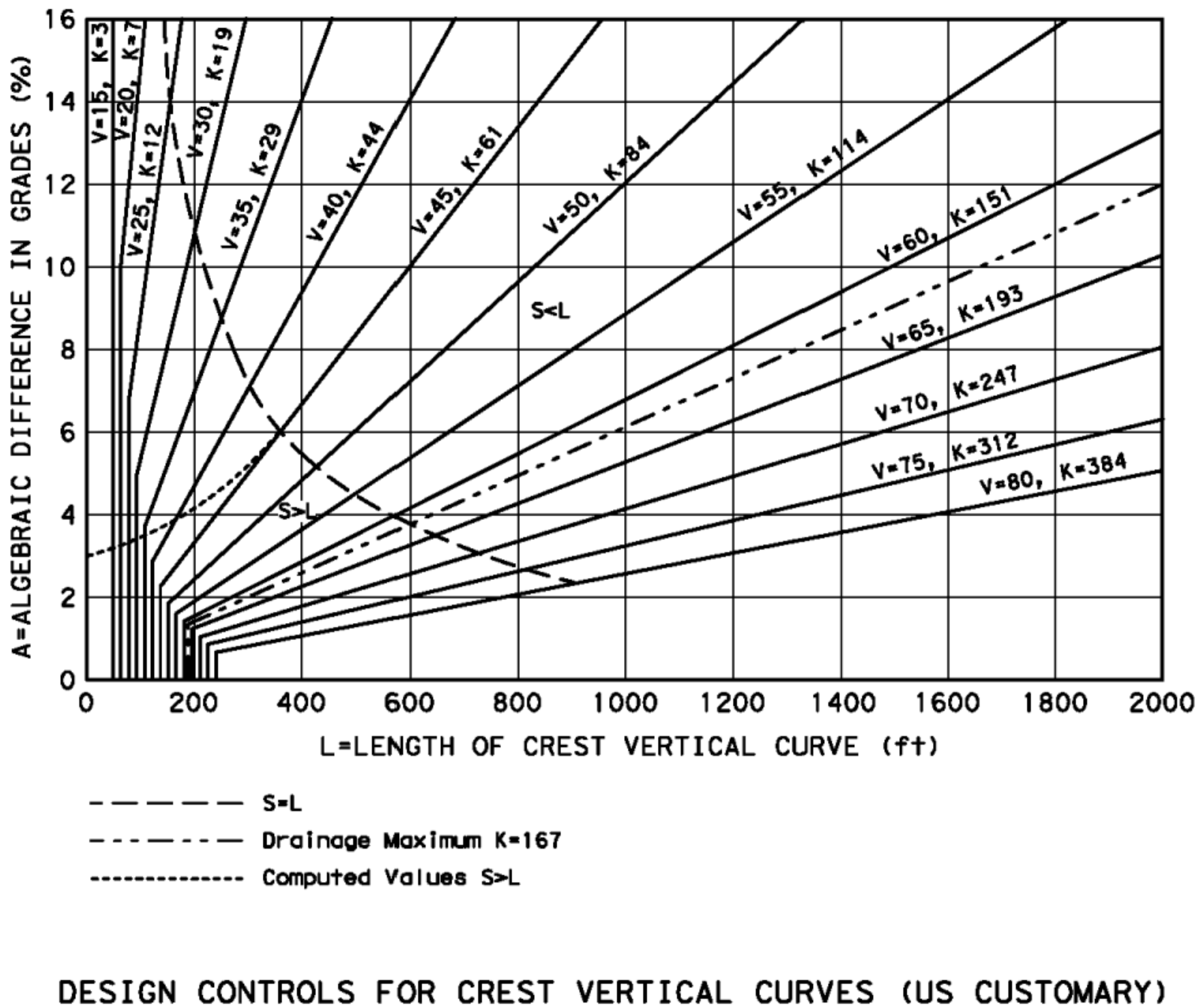


Figure 2-5. (US). Design Controls for Crest Vertical Curves. Click here to see a PDF of the image.

NOTE: Online users can view the metric version of this figure in PDF format.

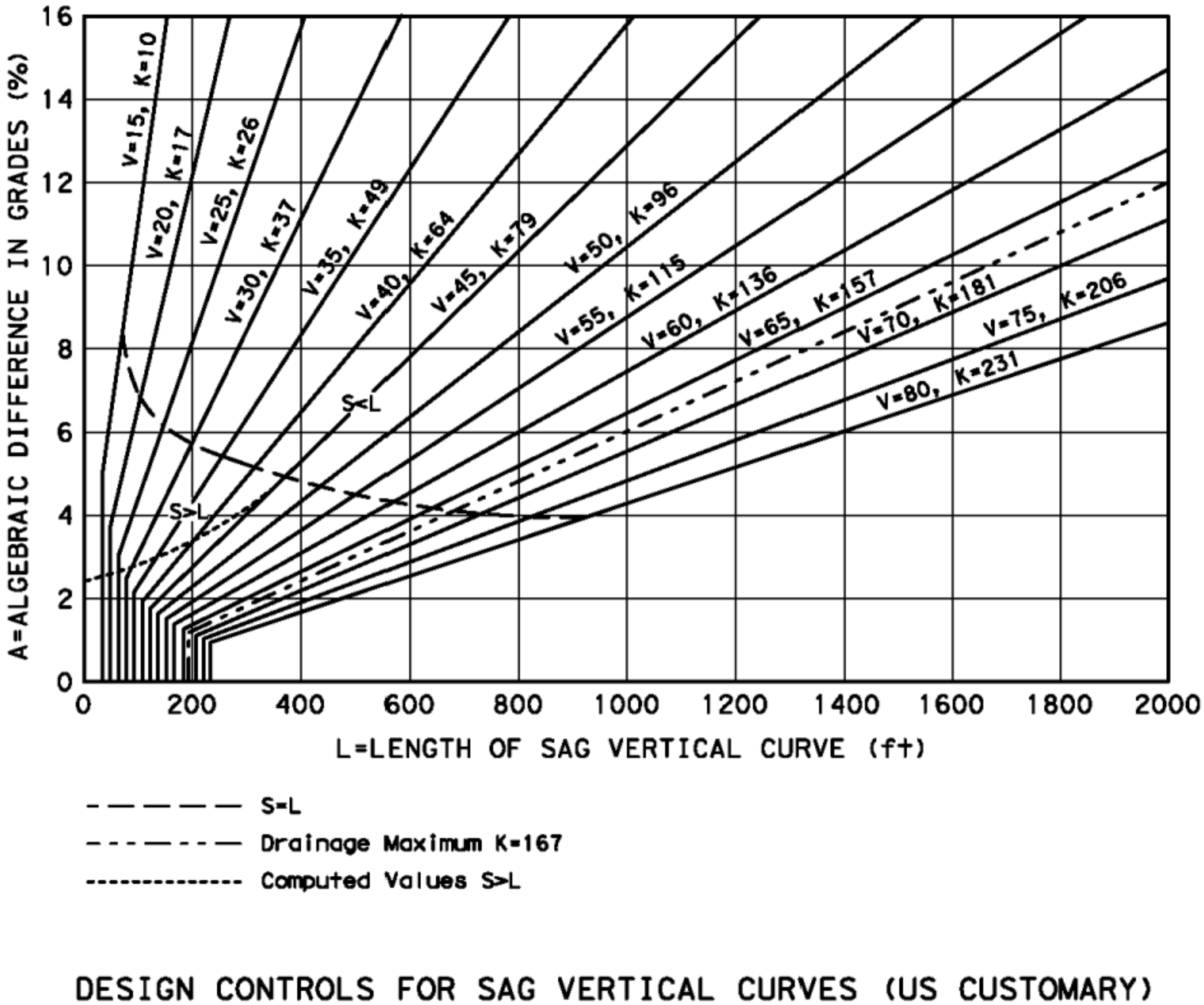


Figure 2-6. (US). Design Controls for Sag Vertical Curves. Click here to see a PDF of the image.

NOTE: Online users can `view the metric version of this figure` in PDF format.

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 economical 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]
- 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
- 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 (i.e., slope of 2.5 percent). In these cases, the inside lanes may be sloped flatter than normal, typically at 1.5 percent but not less than 1.0 percent.

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 to 7 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.
- 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 horizontal clearance 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 or flush medians. 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.

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 accomodations should be considered when a project is scoped. Bicycle consideration is required on urban facilities. To accommodate bicycles, the outside curb lane should be 14 ft [4.2m] from the lane stripe to the gutter joint or gutter lip on a monolithic curb. For a striped bicycle lane, the clear width is 5 ft [1.5m] minimum. For additional guidance, refer to the AASHTO Guide for the Development of Bicycle Facilities.

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 shoulder widths for the various classes of highways are shown in the appropriate subsequent portions of this manual.

Shoulder widths should accomodate bicycle facilities and provide a 1 ft offset to barriers across bridges being replaced or rehabilitated.

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. When any of the following factors are present, 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;
- Full reconstruction or new construction that requires new right-of-way.

In typical suburban development, there are initially few pedestrian trips because there are few closely located pedestrian destinations. However, when pedestrian demand increases with additional development, it may be more difficult and more costly to go back and install 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 Public Accessibility Guidelines for Pedestrian Facilities in the Public Right of Way (PROWAG)* and the *Texas Accessibility Standards* (TAS).

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 2-7(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. For roadways functionally classified as rural, the shoulder may be used to accommodate pedestrian and bicycle traffic. Where a shoulder serves as part of the pedestrian access route, it must meet ADA/TAS requirements.

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 2-7(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 2-8). 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.

Street Crossings. 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.

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 2-9. Curb ramps and crosswalks must be aligned behind the nose of the median island to provide adequate refuge.

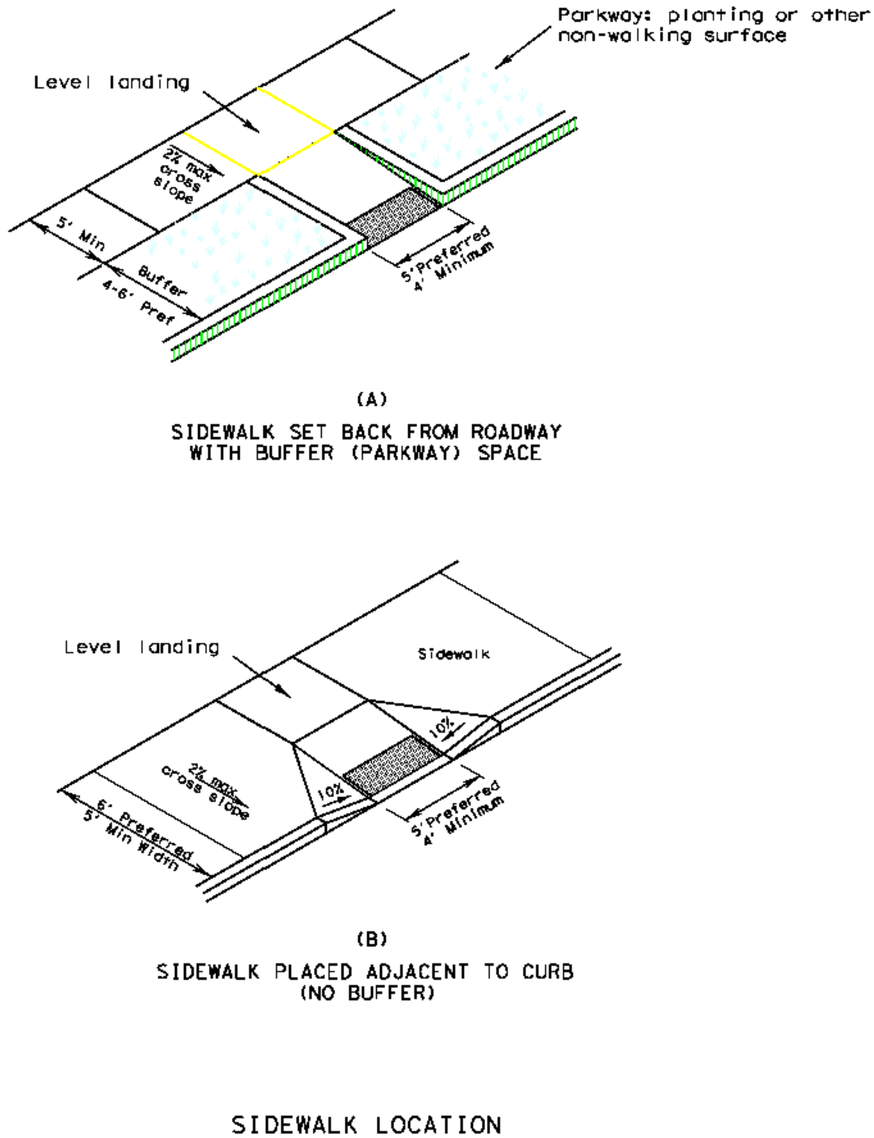


Figure 2-7. `Curb Ramps and Landings`

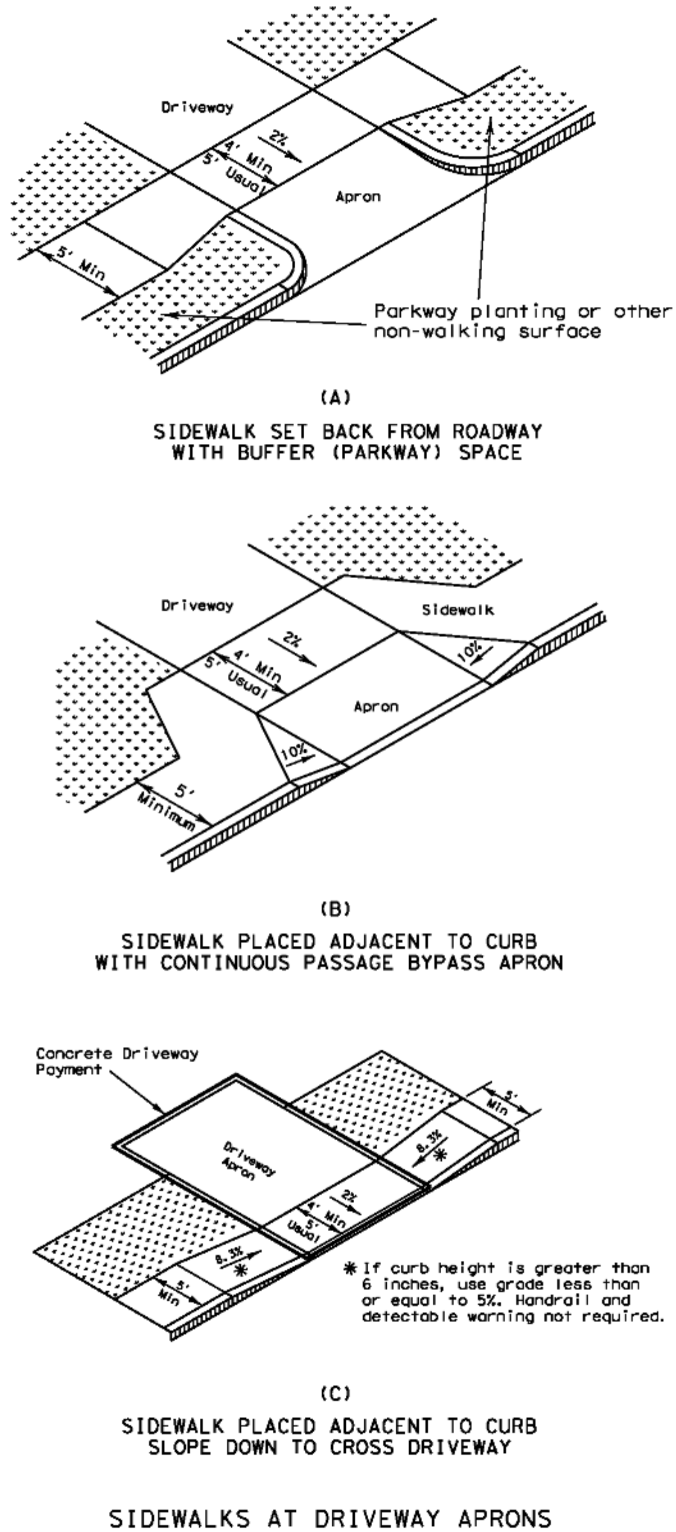
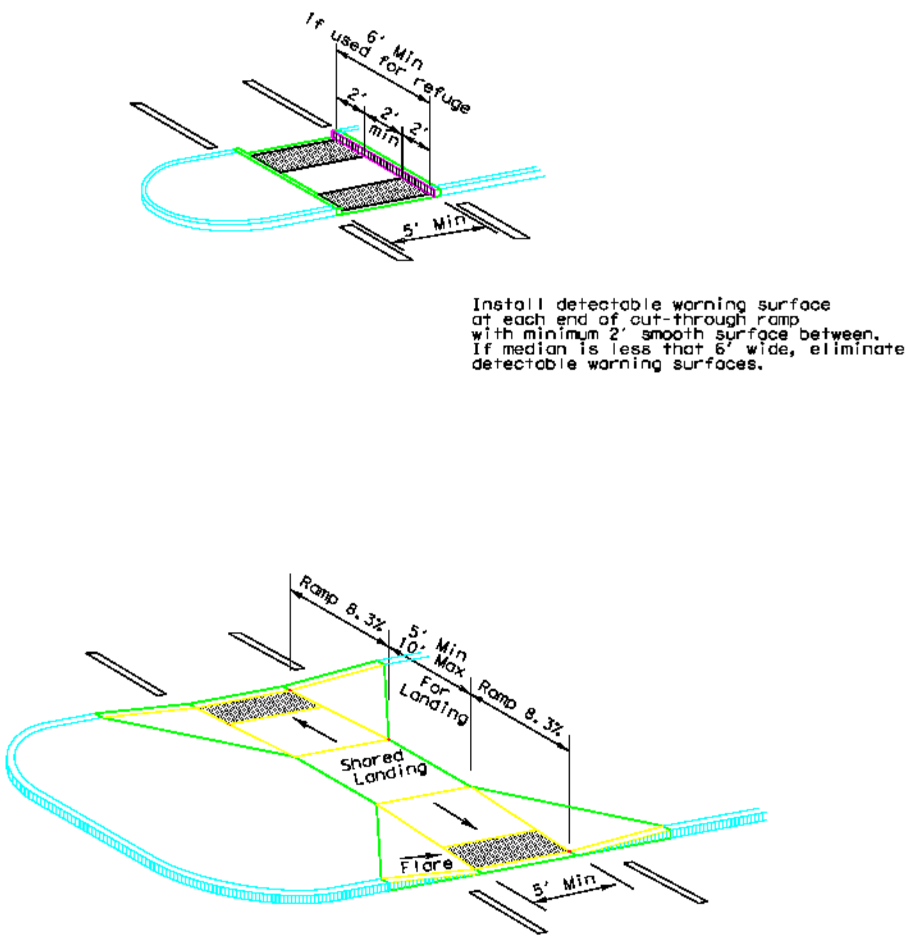


Figure 2-8. Sidewalks at Driveway Aprons.



CURB RAMPS AT MEDIAN ISLANDS

Figure 2-9. Curb Ramps at Median Islands.

NOTE: Online users can view the metric version of this figure.

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
- installation of pavement markings for pedestrian crosswalks

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 accomodate a 5 ft [1.5m] diameter circle and will connect to the continuous passage in each direction of travel as shown in Figure 2-7. 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 should be wholly contained within the markings of the crosswalk. There should 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.

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 2-8. 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 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, bike racks, and parking meters). A clear ground 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 PROWAG/TAS. The clear ground space must have an accessible connection to the sidewalk and must not encroach into the 5 ft [1.5m] minimum sidewalk width by more than 2 ft [610 mm]. Pedestrian push buttons must also be within specified reach ranges of a ground space.

PROWAG/TAS. Specific design requirements to accommodate the needs of persons with disabilities are established by the PROWAG/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.

Curb and Curb and 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 accidents of this nature. The configuration and condition of the roadside greatly affect the extent of damages and injuries for these accidents.

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:
  - Eliminate the obstacle.
  - Redesign the obstacle so that it can by 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

Sideslopes. 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 travelway 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. Accident 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.

  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 treatment 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
- 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 high-speed rural highways. 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, bike 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 shows criteria for clear zones.

Table 2-12: Clear Zones

Location	Functional Classification	Design Speed (mph)	Avg. Daily Traffic	Clear Zone Width (ft)[3,4,5]	
-	-	-	-	Minimum	Desirable
Rural	Freeways	All	All	30 (16 for ramps)	
Rural	Arterial	All	0 - 750 750 - 1500 >1500	10 16 30	16 30 --
Rural	Collector	≥ 50	All	Use above rural arterial criteria.	
Rural	Collector	≤ 45	All	10	--
Rural	Local	All	All	10	--
Suburban	All	All	<8,000	10[6]	10[6]
Suburban	All	All	8,000 - 12,000	10[6]	20[6]
Suburban	All	All	12,000 - 16,000	10[6]	25[6]
Suburban	All	All	>16,000	20[6]	30[6]
Urban	Freeways	All	All	30 (16 for ramps)	
Urban	All (Curbed)	≥ 50	All	Use above suburban criteria insofar as available border width permits.	
Urban	All (Curbed)	≤ 45	All	4 from curb face	6
Urban	All (Uncurbed)	≥ 50	All	Use above suburban criteria.	
Urban	All (Uncurbed)	≤ 45	All	10	--

[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.

[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.

[3] Without barrier or other safety treatment of appurtenances.

[4] Measured from edge of travel lane for all cut sections and for all fill sections where side slopes are 1V:4H or flatter. Where fill slopes are steeper than 1V:4H it is desirable to provide a 10 ft area free of obstacles beyond the toe of slope.

[5] Desirable, rather than minimum, values should be used where feasible.

[6] Purchase of 5 ft or less of additional right-of-way strictly for satisfying clear zone provisions is not required.

NOTE: Online users can view the metric version of this table in PDF format.

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

Section 7
Drainage Facility Placement

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.).

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 2-13: Maximum Diameter of Small Pipe Culvert

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

NOTE: Click here to see the metric version of the 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 ft^2 [2.3 m^2].

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, unless it is bridge class culvert.
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 ft^2 [2.3 m^2].

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, unless it is bridge class
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 shall be, in order of preference, safety treated, shielded with guardrail and/or bridge rail on the approach and across the culvert. Table 2-14 provides guidelines for installing guardrails and barrier rails. Recommended treatment options are in the following priority:

1. Safety treat culvert ends.
2. Meet clear zone requirements, unless it is bridge class.

3. Shield with appropriate barrier or attenuator.

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

Depth of Cover	Treatment
Less than 9 in.	Bridge Railing
9 in. but less than 36 in.	Steel post welded to base plate and bolted to culvert ceiling (Low-fill culvert post)
36 in. or more	Guardrail

NOTE: Online users can view the metric version of this table in PDF format

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 2-10.

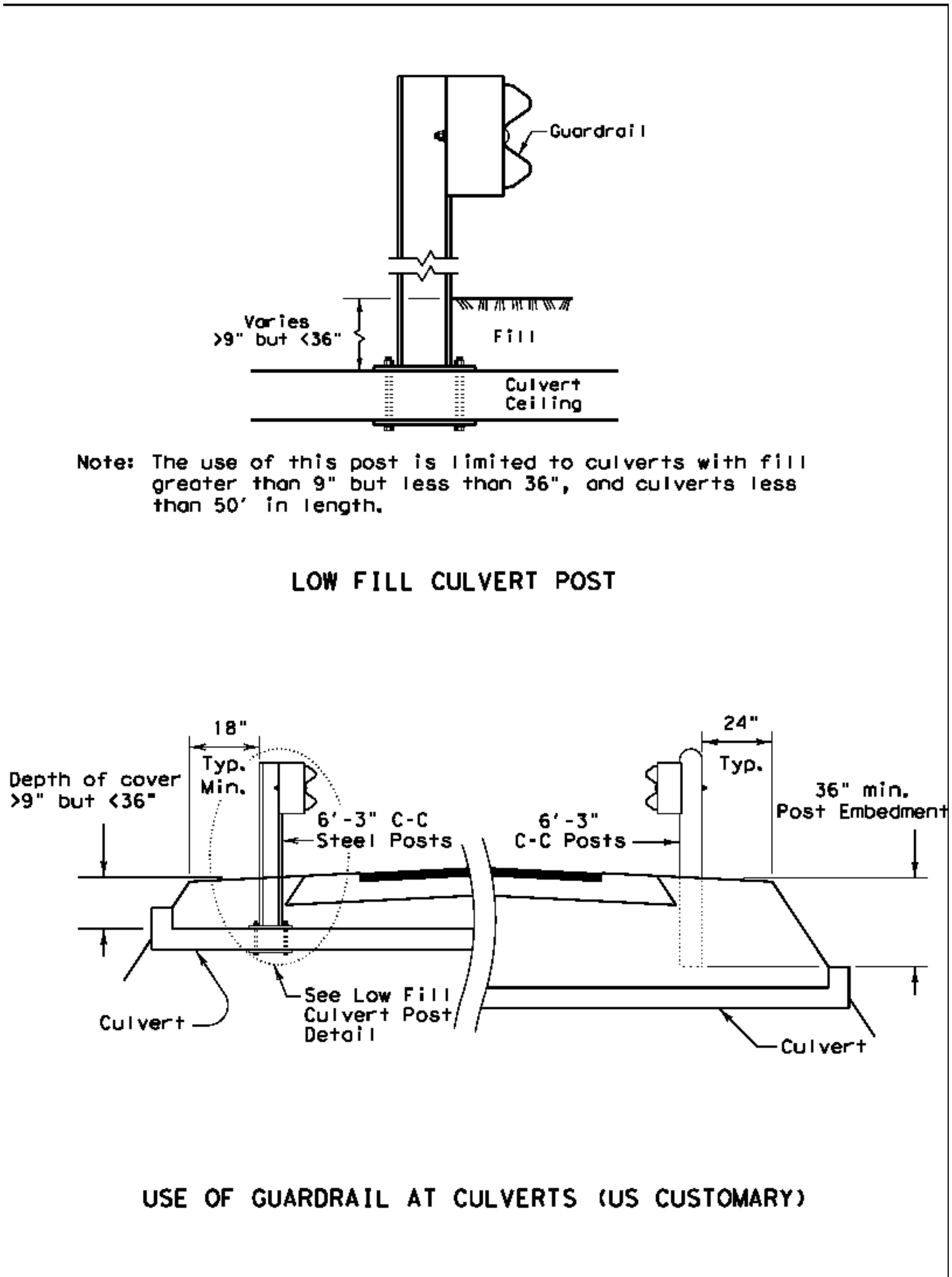


Figure 2-10. Use of Guardrail at Culverts. Click here to see a PDF of the image.

Online users can view the metric version of this figure.

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 2-11.
- 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 2-12. `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.

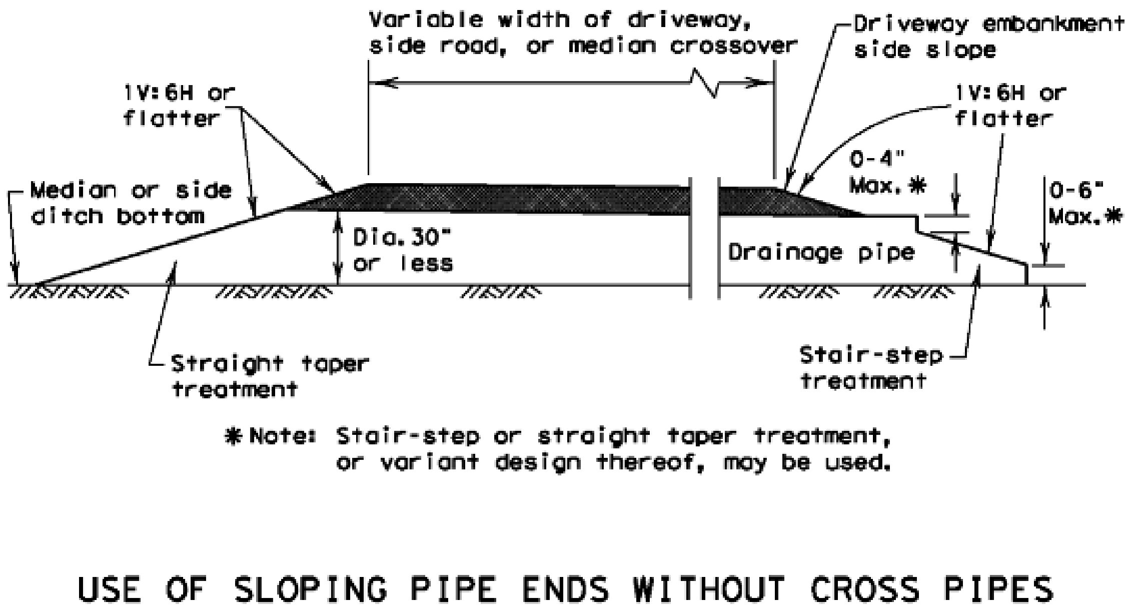


Figure 2-11. Use of Sloping Pipe Ends Without Cross Pipes.

NOTE: Online users can view the metric PDF of this figure.

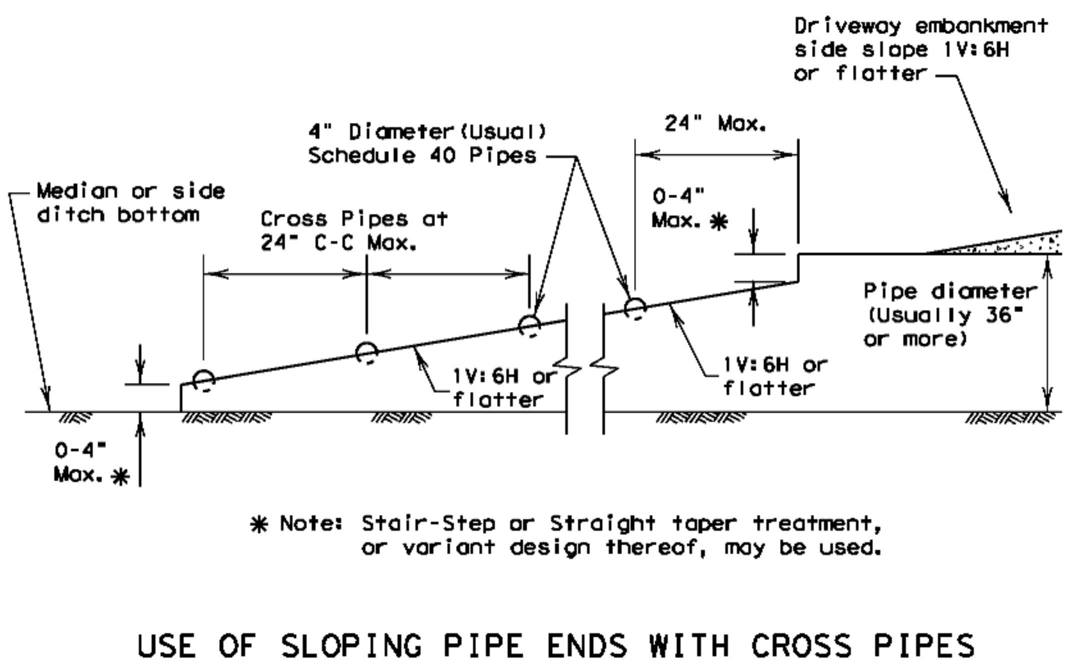


Figure 2-12. Use of Sloping Pipe Ends With Grates.

NOTE: Online users can view the metric version of this figure.

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 horizontal clearance requirements, other ditch configurations may be used. Typically, guardrail is not necessary where the preferred ditch cross sections are provided.

Table 2-15: Preferred Ditch Cross Sections

-	Preferred Maximum Back Slope (Vertical:Horizontal)	
Given Front Slope (Vertical:Horizontal)	V-Shaped	Trapezoidal-Shaped
1V:8H	1V:3.5H	1V:2.5H
1V:6H	1V:4H	1V:3H
1V:4H	1V:6H	1V:4H
1V:3H	Level	1V:8H

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. If these conditions are not met, then a design exception or design waiver for the intersecting roadway will be appropriate. Existing geometry will include all cross sectional elements. The definition of intersecting roadways excludes driveways.

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

Contents:

Section 1 — Overview

Introduction

This chapter presents guidelines that are applicable to all new location and reconstruction projects for several different classes of roadways including the following:

- urban streets
- suburbanways
- two-lane rural highways
- multilane rural highways
- freeways.

Departures from these guidelines are governed in Design Exceptions, Design Waivers and Design Variances, 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 for these facilities is controlled only through driveway locations and medians.

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 minmum 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

(US Customary)			
Item	Functional Class	Desirable	Minimum
Design Speed (mph)	All	Up to 60	30
Minimum Horiz. Radius	All	See Tables 2-3 and 2-4, Figure 2-2	
Maximum Gradient (%)	All	See Table 2-9	
Stopping Sight Distance	All	See Table 2-1	
Width of Travel Lanes (ft)	Arterial Collector Local	12 12 11-12	11[1] 10[2] 10[2,3]
Curb Parking Lane Width (ft)	Arterial Collector Local	12 10 9	10[4] 7[5] 7[5]
Shoulder Width[6] (ft), Uncurbed Urban Streets	Arterial Collector Local	10 8 --	4 3 2
Width of Speed Change Lanes (ft)	Arterial and Collector Local	11-12 10-12	10 9
Offset to Face of Curb (ft)	All	2	1
Median Width	All	See Medians	
Border Width (ft)	Arterial Collector	20 20	15 15
Right-of-Way Width	All	Variable [7]	
Clear Sidewalk Width (ft)[10]	All	6-8[8]	5
On-Street Bicycle Lane Width	All	See Chapter 6, Bicycle Facilities	
Superelevation	All	See Chapter 2, Superelevation	
Horizontal Clearance Width	All	See Table 2-11	
Vertical Clearance for New Structures (ft)	All	16.5	16.5[9]
Turning Radii	-	See Chapter 7, Minimum Designs for Truck and Bus Turns	

Table 3-1: Geometric Design Criteria for Urban Streets

(US Customary)
[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.

Table 3-1: Geometric Design Criteria for Urban Streets

(Metric)			
Item	Functional Class	Desirable	Minimum
Design Speed (km/h)	All	Up to 100	50
Minimum Horiz. Radius	All	See Tables 2-3 and 2-4, Figure 2-2	
Maximum Gradient (%)	All	See Table 2-9	
Stopping Sight Distance	All	See Table 2-1	
Width of Travel Lanes (m)	Arterial Collector Local	3.6 3.6 3.3-3.6	3.3[1] 3.0[2] 3.0[2,3]
Curb Parking Lane Width (m)	Arterial Collector Local	3.6 3.0 2.7	3.0[4] 2.1[5] 2.1[5]
Shoulder Width 6 (m), Uncurbed Urban Streets	Arterial Collector Local	3.0 2.4 --	1.2 0.9 0.6
Width of Speed Change Lanes (m)	Arterial and Collector Local	3.3-3.6 3.0-3.6	3.0 2.7
Offset to Face of Curb (m)	All	0.6	0.3
Median Width	All	See Medians	
Border Width (m)	Arterial Collector	6.0 6.0	4.5 4.5
Right-of-Way Width	All	Variable 7	
Clear Sidewalk Width (m)[10]	All	1.8-2.4[8]	1.5

Table 3-1: Geometric Design Criteria for Urban Streets

(Metric)			
On-Street Bicycle Lane Width	All	See Chapter 6, Bicycle Facilities	
Superelevation	All	See Chapter 2 Superelevation	
Horizontal Clearance Width	All	See Table 2-11	
Vertical Clearance for New Structures (m)	All	5.0	5.0[9]
Turning Radii	-	See Chapter 7, Minimum Designs for Truck and Bus Turns	

[1] In highly restricted locations or locations with few trucks and speeds less than or equal to 60 km/h 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] In residential areas, 2.7 m minimum.
[4] Where there is no demand for use as a future through lane, 2.4 m minimum.
[5] In commercial and industrial areas, 2.4 m 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 5.0 m but never less than 4.4 m. Existing structures that provide at least 4.3 m 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.

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 intersection.

  Medians used on urban streets include the following types:

- raised

- flush
- 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
- 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. For maintenance considerations in preventing recurring damage to the divider, the divider should be at least 2 ft [0.6 m]. If pedestrians are expected to cross the divider, then the divider should be a minimum of 5 ft [1.5 m] wide in order to accommodate a cut-though landing or refuge area that is at least 5 ft x 5 ft [1.5 m x 1.5 m] cut-through landing or refuge area. 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
- 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 accident 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

(US Customary)				(Metric)		
Design Speed Mph	Width of TWLTL - ft			Design Speed Km/h	Width of TWLTL – m	
	Desirable	Minimum			Desirable	Minimum
Less than or equal to 40	12 - 14	11		Less than or equal to 60	3.6 - 4.2	3.3
45 - 50	14	12		70 - 80	4.2	3.6
Greater than 50	16	14		Greater than 80	4.8	4.2

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
- 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. Seven-lane 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.

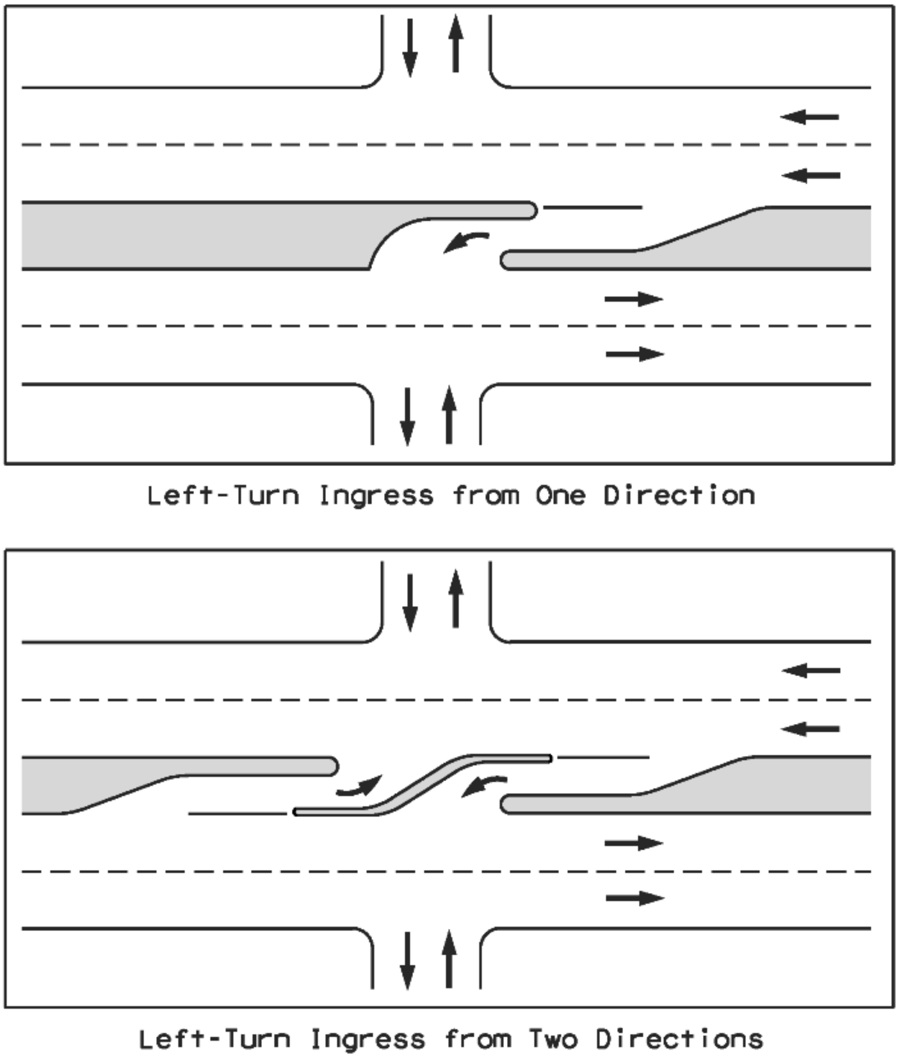


Figure 3-1. Types of Directional Openings. Click here to see a PDF of the image.

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 inTable 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 horizontal clearance 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, shoulders or parking lanes, median, borders, 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).

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

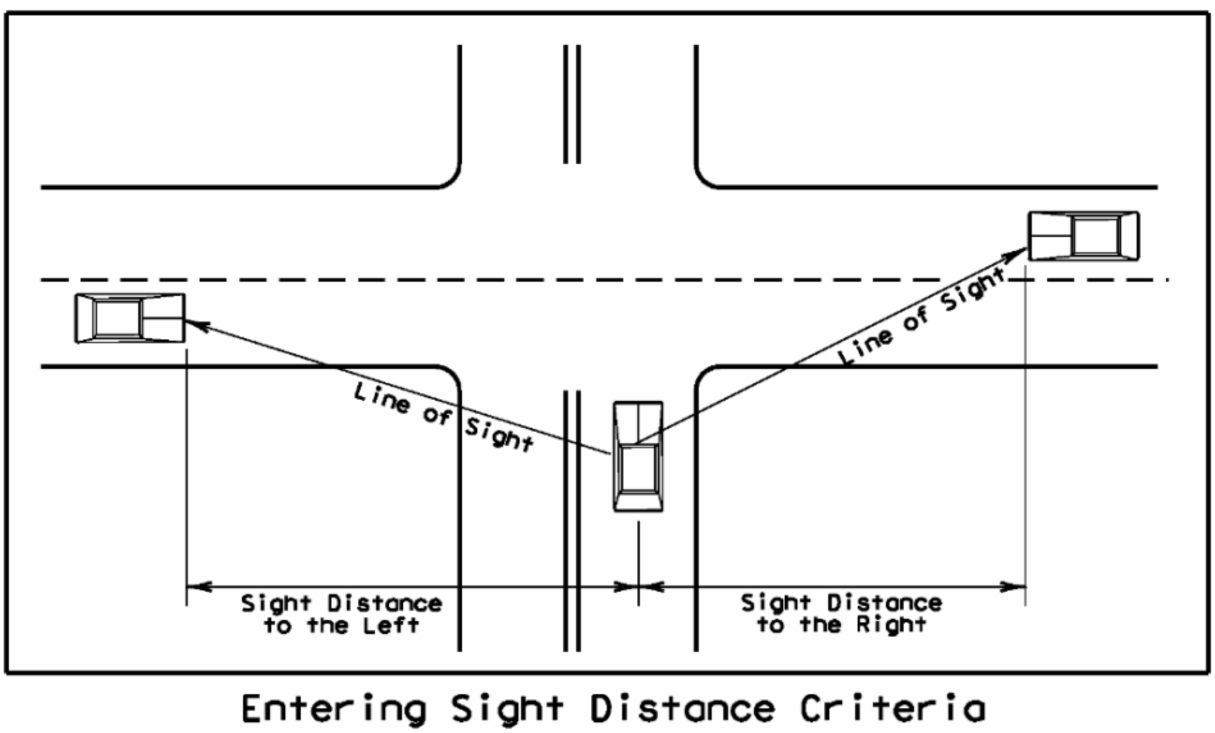


Figure 3-2. Entering Intersection Lines of Sight. Click here to see a PDF of the image.

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 at least 5 ft. [1.5 m] wide, preferably at least 6 ft. [1.8 m]. 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.

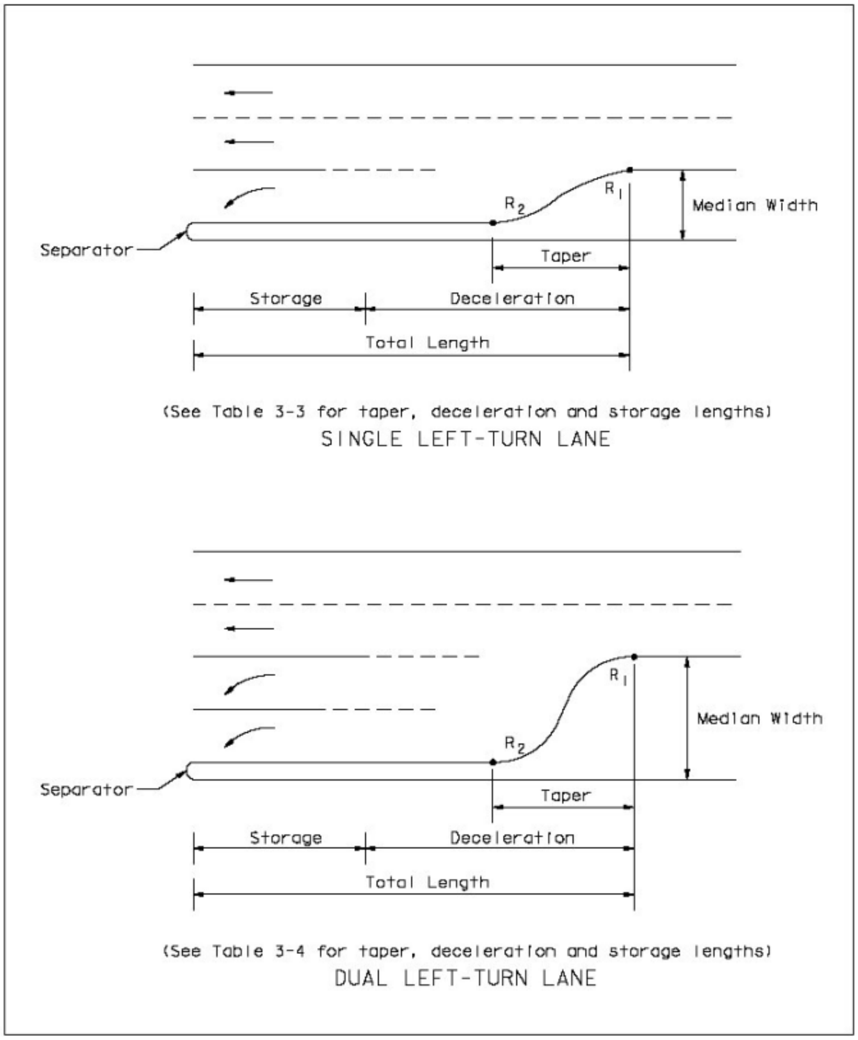


Figure 3-3. Left-Turn Lanes on Urban Streets. Click here to see a PDF of the image.

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[1]

(US Customary)						
Speed (mph)	Deceleration Length[2](ft)	Taper Length (ft)	Storage Length (ft)			
	-		Signalized		Non-Signalized	
	-		Calculated	Minimum[4]	Calculated[5]	Minimum[4]
30	160	50	See footnote 3	100	See footnote 5	100

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

(US Customary)						
35	215	50	See footnote 3	100	See footnote 5	100
40	275	50	See footnote 3	100	See footnote 5	100
45	345	100	See footnote 3	100	See footnote 5	100
50	425	100	See footnote 3	100	See footnote 5	100
55	510	100	See footnote 3	100	See footnote 5	100
(Metric)						
Speed (km/h)	Deceleration Length[2] (m)	Taper Length (m)	Storage Length (m)			
			Signalized		Non-Signalized	
			Calculated	Minimum[4]	Calculated	Minimum[4]
50	50	15	See footnote 3	30	See footnote 5	30
60	65	15	See footnote 3	30	See footnote 5	30
70	85	30	See footnote 3	30	See footnote 5	30
80	105	30	See footnote 3	30	See footnote 5	30
90	130	30	See footnote 3	30	See footnote 5	30

[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 = (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 (15km/h) before clearing the through traffic lane. See Table 3-3A.

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

US Customary (ft)			**Metric (m)**		
Speed	Speed Differential*		Speed	Speed Differential*	
(mph)	15 mph	20 mph	(km/h)	20 km/h	25km/h
30	110	75	50	40	35
35	160	110	60	60	50
40	215	160	70	75	65
45	275	215	80	95	85
50	345	275	90	115	105
55	425	345			
* 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. [3m]) 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)
<5	25	7.6
5-9	30	9.1

10-14	35	10.7
15-19	40	12.2

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 [1]

(US Customary)				
Speed	Deceleration	Taper	Storage Length (ft)	
(mph)	Length[2](ft)	Length (ft)	Calculated [3]	Minimum[4]
30	160	100	See footnote 3	100
35	215	100	See footnote 3	100
40	275	100	See footnote 3	100
45	345	150	See footnote 3	100
50	425	150	See footnote 3	100
55	510	150	See footnote 3	100
Table 3-4: Lengths of Dual Left-Turn Lanes on Urban Streets				
(Metric)				
Speed	Deceleration	Taper	Storage Length (m)	
(km/h)	Length[2](m)	Length (m)	Calculated[3]	Minimum[4]
50	50	30	See footnote 3	30
60	65	30	See footnote 3	30
70	85	45	See footnote 3	30
80	105	45	See footnote 3	30
90	130	45	See footnote 3	30
See Table 3-3 for footnotes.				

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.

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.

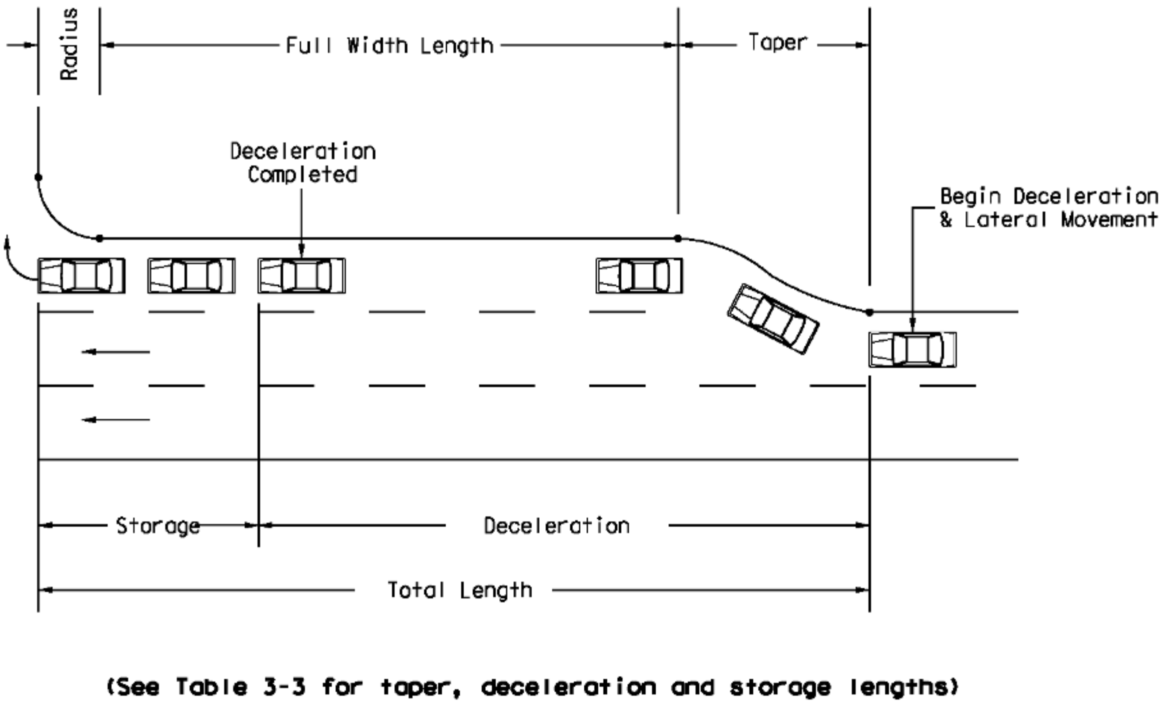


Figure 3-4. Lengths of Right-Turn Deceleration Lanes. Click here to see a PDF of the image.

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.

Section 3 — Suburban Roadways

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
- Horizontal Clearances
- Borders
- Grade Separations and Interchanges
- Intersections
- 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

(US Customary)			
Item	Functional Class	Desirable	Minimum

Table 3-5: Geometric Design Criteria for Suburban Roadways

(US Customary)			
Design Speed (mph)	All	60	50
Minimum Horizontal Radius	All	See Tables 2-3 and 2-4	
Maximum Gradient (%)	All	See Table 2-9	
Stopping Sight Distance	All	See Table 2-1	
Width of Travel Lanes (ft)	Arterial Collector	12 12	11[1] 10[2]
Curb Parking Lane Width (ft)	All	None	
Shoulder Width (ft)	All	10	4
Width of Speed Change Lanes[3](ft)	All	11-12	10
Offset to Face of Curb (ft)	All	2	1
Median Width	All	See Medians, Urban Streets	
Border Width (ft)	Arterial Collector	20 20	15 15
Right-of-Way Width (ft)	All	Variable[4]	
Sidewalk Width (ft)	All	6-8[5]	5
Superelevation	All	See Chapter 2, Superelevation	
Horizontal Clearance	All	See Table 2-11	
Vertical Clearance for New Strs. (ft)	All	16.5	16.5[6]
Turning Radii	All	See Chapter 7, Minimum Designs for Truck and Bus Turns	

[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.

Table 3-5: Geometric Design Criteria for Suburban Roadways

(Metric)			
Item	Functional Class	Desirable	Minimum
Design Speed (km/h)	All	100	80

Table 3-5: Geometric Design Criteria for Suburban Roadways

(Metric)			
Minimum Horizontal Radius	All	See Tables 2-3 and 2-4	
Maximum Gradient (%)	All	See Table 2-9	
Stopping Sight Distance	All	See Table 2-1	
Width of Travel Lanes (m)	Arterial Collector	3.6 3.6	3.3[1] 3.0[2]
Curb Parking Lane Width (m)	All	None	
Shoulder Width (m)	All	3.0	1.2
Width of Speed Change Lanes[3] (m)	All	3.3-3.6	3.0
Offset to Face of Curb (m)	All	0.6	0.3
Median Width	All	See Medians, Urban Streets	
Border Width (m)	Arterial Collector	6.0 6.0	4.5 4.5
Right-of-Way Width (m)	All	Variable[4]	
Sidewalk Width (m)	All	1.8-2.4[5]	1.5
Superelevation	All	See Chapter 2, Superelevation	
Horizontal Clearance	All	See Table 2-11	
Vertical Clearance for New Strs. (m)	All	5.0	5.0[6]
Turning Radii	All	See Chapter 7, Minimum Designs for Truck and Bus Turns	

[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.

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 accidents is increased due to the high-speed differentials. Driver expectancy is also violated because through traffic traveling at high speeds does 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 accidents. In addition, accident 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.
- 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, shoulders, median, sidewalks, and borders.

Horizontal Clearances

Table 2-11: Horizontal Clearances presents the general horizontal clearance 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

(US Customary)					
Functional	-	Minimum Design Speed (mph) for future ADT of:			
Class	Terrain	< 400	400-1500	1500-2000	> 2000
Arterial	Level Rolling	70 60			
Collector	Level Rolling	50[1] 40[2]	50 40	50 40	60 50
Local[3]	Level Rolling	40[2] 30	50 40	50 40	50 40
(Metric)					
Functional	-	Minimum Design Speed (km/h) for future ADT of:			
Class	Terrain	< 400	400-1500	1500-2000	> 2000
Arterial	Level Rolling	110 100			
Collector	Level Rolling	80[1] 60[2]	80 60	80 60	100 80

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

(US Customary)					
Local[3]	Level Rolling	60[2] 50	80 60	80 60	80 60

[1] A 40 mph [60 km/h] minimum design speed may be used where roadside environment or unusual design considerations dictate.
[2] A 30 mph [50 km/h] minimum design speed may be used where roadside environment or unusual design considerations dictate.
[3] Applicable only to off-system routes that are not functionally classified at a higher classification.

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

(US Customary)		
Geometric Design Element	Functional Class	Reference or Design Value
Design Speed	All	Table 3-6
Minimum Horizontal Radius	All	Table 2-3and Table 2-4
Max. Gradient	All	Table 2-9
Stopping Sight Distance	All	Table 2-1
Width of Travel Lanes	All	Table 3-8
Width of Shoulders	All	Table 3-8
Vertical Clearance, New Structures	All	16.5 ft[1]
Horizontal Clearance	All	Table 2-11
Pavement Cross Slope	All	Chapter 2, Pavement Cross Slope
(Metric)		
Geometric Design Element	Functional Class	Reference or Design Value
Design Speed	All	Table 3-6
Minimum Horizontal Radius	All	Table 2-3and Table 2-4
Max. Gradient	All	Table 2-9
Stopping Sight Distance	All	Table 2-1
Width of Travel Lanes	All	Table 3-8
Width of Shoulders	All	Table 3-8
Vertical Clearance, New Structures	All	5.0 m[1]
Horizontal Clearance	All	Table 2-11

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

(US Customary)		
Pavement Cross Slope	All	Chapter 2, Pavement Cross Slope
[1] 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

(US Customary)					
Functional Class	Design Speed (mph)	Minimum Width [1,2](ft) for future ADT of:			
-	-	< 400	400-1500	1500-2000	> 2000
Arterial	LANES (ft)				
-	All	12			
-	SHOULDERS (ft)				
-	All	4[3]	4[3] or 8[3]	8[3]	8 - 10[3]
Collector	LANES (ft)				
-	30	10	10	11	12
-	35	10	10	11	12
-	40	10	10	11	12
-	45	10	10	11	12
-	50	10	10	12	12
-	55	10	10	12	12
-	60	11	11	12	12
-	65	11	11	12	12
-	70	11	11	12	12
-	75	11	12	12	12
-	80	11	12	12	12
-	SHOULDERS (ft)				
-	All	2[4,5]	4[5]	8[5]	8 - 10[5]
Local6	LANES (ft)				
-	30	10	10	11	12
-	35	10	10	11	12
-	40	10	10	11	12
-	45	10	10	11	12
-	50	10	10	11	12
-	SHOULDERS (ft)				
-	All	2	4	4	8

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

(US Customary)

[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.

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

(Metric)					
Functional Class	Design Speed (km/h)	Minimum Width [1,2](m) for future ADT of:			
-	-	< 400	400-1500	1500-2000	> 2000
Arterial	LANES (m)				
-	All	3.6			
-	SHOULDERS (m)				
-	All	1.2[3]	1.2[3] or 2.4[3]	2.4[3]	2.4 - 3.0[3]
Collector	LANES (m)				
-	50	3.0	3.0	3.3	3.6
-	60	3.0	3.0	3.3	3.6
-	70	3.0	3.0	3.3	3.6
-	80	3.0	3.0	3.6	3.6
-	90	3.0	3.0	3.6	3.6
-	100	3.3	3.3	3.6	3.6
-	110	3.3	3.3	3.6	3.6
-	120	3.3	3.6	3.6	3.6
-	130	3.3	3.6	3.6	3.6
-	SHOULDERS (m)				
-	All	0.6[4,5]	1.2[5]	2.4[5]	2.4-3.0[5]
Local[6]	LANES (m)				
-	50	3.0	3.0	3.3	3.6
-	60	3.0	3.0	3.3	3.6
-	70	3.0	3.0	3.3	3.6
-	80	3.0	3.0	3.3	3.6
-	SHOULDERS (m)				
-	All	0.6	1.2	1.2	2.4

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

(Metric)
[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.

The following notes apply to Table 3-8:

- 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

(US Customary)				
Functional Class	Roadway Clear Width [1] (ft) for ADT of:			
-	< 400	400-1500	1500-2000	> 2000
Local	20	22	24	28
Collector	22	22	24	28
Arterial	Traveled Way + 6 ft			
(Metric)				
Functional Class	Roadway Clear Width [1] (m) for ADT of:			
-	< 400	400-1500	1500-2000	> 2000
Local	6.0	6.6	7.2	8.4
Collector	6.6	6.6	7.2	8.4
Arterial	Traveled Way + 1.8 m			
[1] Clear width between curbs or rails, whichever is lesser, is considered to be at least the same as 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
- 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 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
- 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)		
K-Values for Determining Length of Crest Vertical Curve for Various Passing Sight Distances		
Design Speed (mph)	Minimum Passing Sight Distance for Design (ft)	K-Value[1]
20	710	180
25	900	289
30	1090	424
35	1280	585
40	1470	772
45	1625	943
50	1835	1203
55	1985	1407
60	2135	1628
65	2285	1865
70	2480	2197
75	2580	2377
80	2680	2565
(Metric)		
K-Values for Determining Length of Crest Vertical Curve for Various Passing Sight Distances		
Design Speed (km/h)	Minimum Passing Sight Distance for Design (m)	K-Value[1]
30	200	46
40	270	84
50	345	138
60	410	195
70	485	272
80	540	338
90	615	438
100	670	520
110	730	617
120	775	695
130	815	769

Table 3-10: Passing Sight Distance

(US Customary)
[1]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.

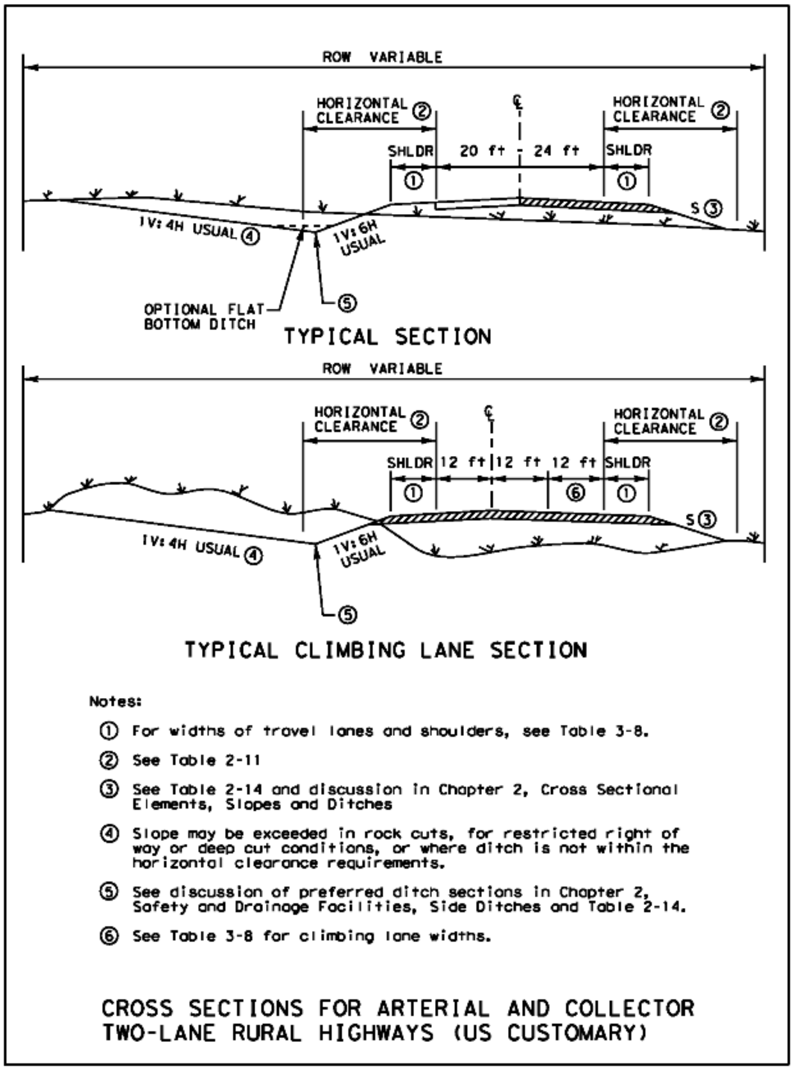


Figure 3-5. (US). Cross Sections for Arterial and Collector Two-Lane Rural Highways. Click US Customary or Metric to see a PDF of the image.

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 accident costs. Figure 3-6 provides recommendations for when left-turn lanes should be considered based on traffic volumes.

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 3-13.

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.

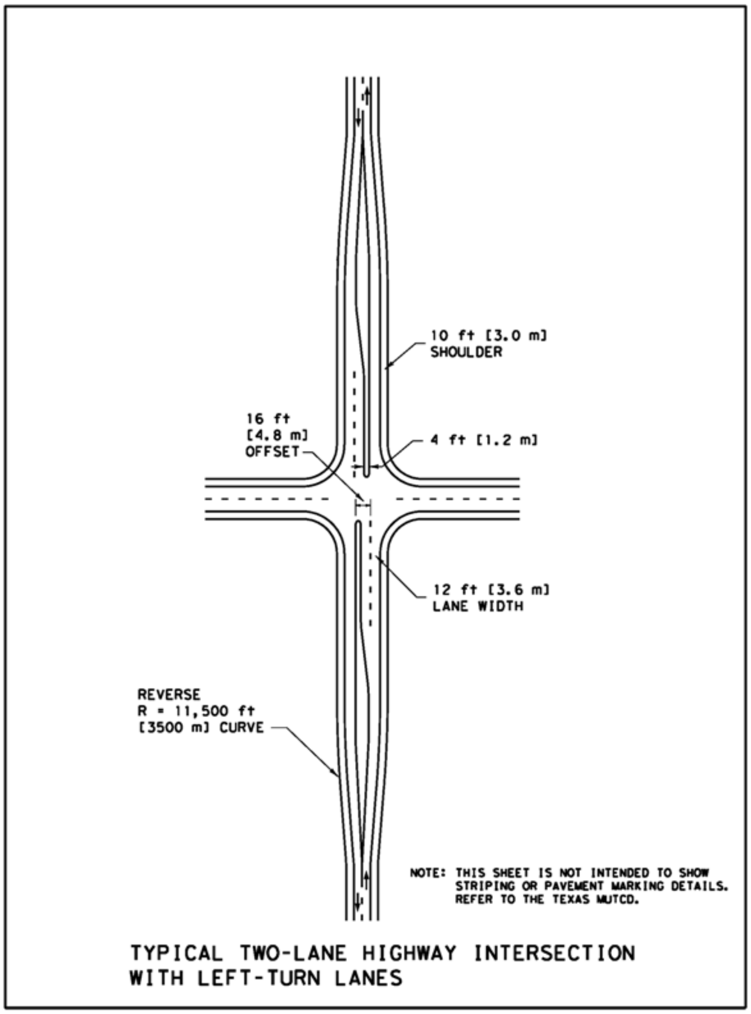


Figure 3-6. Typical Two-Lane Highway Intersection with Left-Turn Lanes. Click here to see a PDF of the image.

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

Opposing Volume (vph)	Advancing Volume (vph)			
-	5 % Left Turns	10 % Left Turns	20 % Left Turns	30 % Left Turns
40 mph [60 km/h] Design Speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50 mph [80 km/h] Design Speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
60 mph [100 km/h] Design Speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

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 lane is being constructed in addition to the through lanes and shoulders, the minimum right turn 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.

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 3-8 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 cross at approximately right angles. Where crossroad skew is flatter than 60 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 3-6 and Figure 3-7. The guidelines apply for all functional classes of roadways.

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

(US Customary)							
Type of Facility		Six-Lane Divided		Four-Lane Divided		Four-Lane Undivided1	
Design Speed (Arterials)[2] (mph)		Min.		Min.		Min.	
Flat		70[3]		70[3]		70[3]	
Rolling		60[4]		60[4]		60[4]	
Lane Width (ft)		12					
-		Des.	Min.	Des.	Min.	Des.	Min
Median Width (ft)	Surfaced	16	4	16	4	Not Applicable	
-	Depressed	76	48	76	48	-	
Shoulder Outside (ft)		10	8[5]	10	8[5]	10	85
Shoulder Inside (ft) for Depressed Medians		10	4	4	4	Not applicable	
Min. Structure Widths for Bridges to Remain in place (ft)	Depressed Median	--	42	--	30	--	56

[1] Undivided section may be used on two-lane highways to improve passing opportunities. Most appropriate for use in rolling terrain and/or restricted right of way conditions.

[2] For multilane collectors, minimum design speed values are 10 mph less than tabulated.

[3] 60 mph acceptable for heavy betterment under unusual circumstances. Otherwise, 70 mph should be minimum.

[4] 50 mph acceptable for heavy betterment under unusual circumstances. Otherwise, 60 mph should be minimum for rural design.

[5] 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.

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

(Metric)							
Type of Facility -		Six-Lane Divided		Four-Lane Divided		Four-Lane Undivided[1]	
Design Speed (Arterials)[2] (km/h)		Min.		Min.		Min.	
Flat		110[3]		110[3]		110[3]	
Rolling		100[4]		100[4]		100[4]	
Lane Width (m)		3.6					
-		Des.	Min.	Des.	Min.	Des.	Min.
- Median Width (m)	Surfaced	4.8	1.2	4.8	1.2	Not applicable	
-	Depressed -	22.8	14.4	22.8	14.4	-	
Shoulder Outside (m)		3.0	2.4[5]	3.0	2.4[5]	3.0	2.4[5]
Shoulder Inside (m) for Depressed Medians		3.0	1.2	1.2	1.2	Not applicable	
Min. Structure Widths for Bridges to Remain in place (m)	Depressed Median	--	12.6	--	9.0	--	16.8

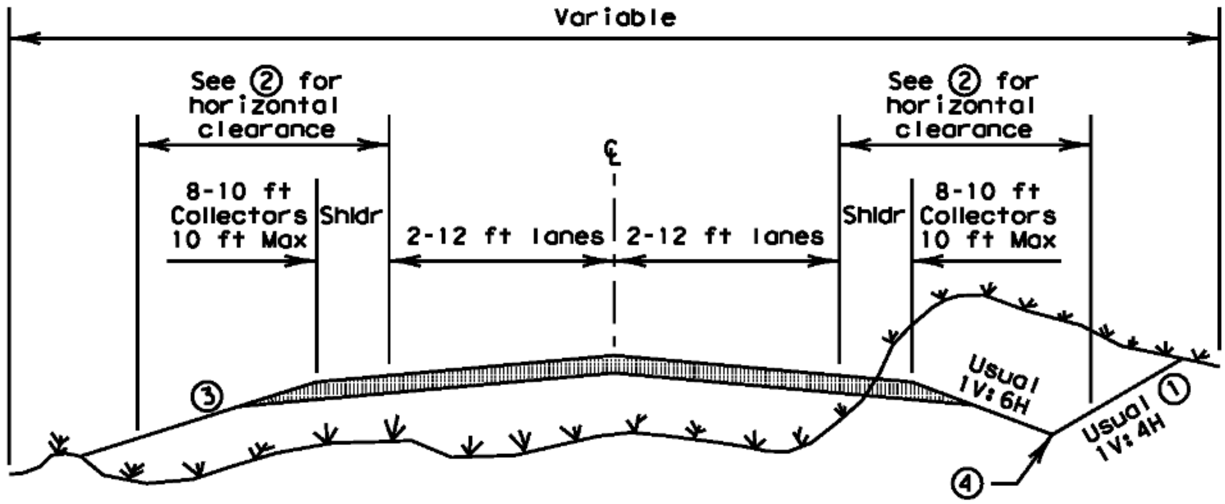
[1] Undivided section may be used on two-lane highways to improve passing opportunities. Most appropriate for use in rolling terrain and/or restricted right of way conditions.

[2] For multilane collectors, minimum design speed values are 20 km/h less than tabulated.

[3] 100 km/h acceptable for heavy betterment under unusual circumstances. Otherwise, 110 km/h should be minimum.

[4] 80 km/h acceptable for heavy betterment under unusual circumstances. Otherwise, 100 km/h should be minimum for rural design.

[5] 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.



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. click US Customary or Metric to see a PDF of the image.

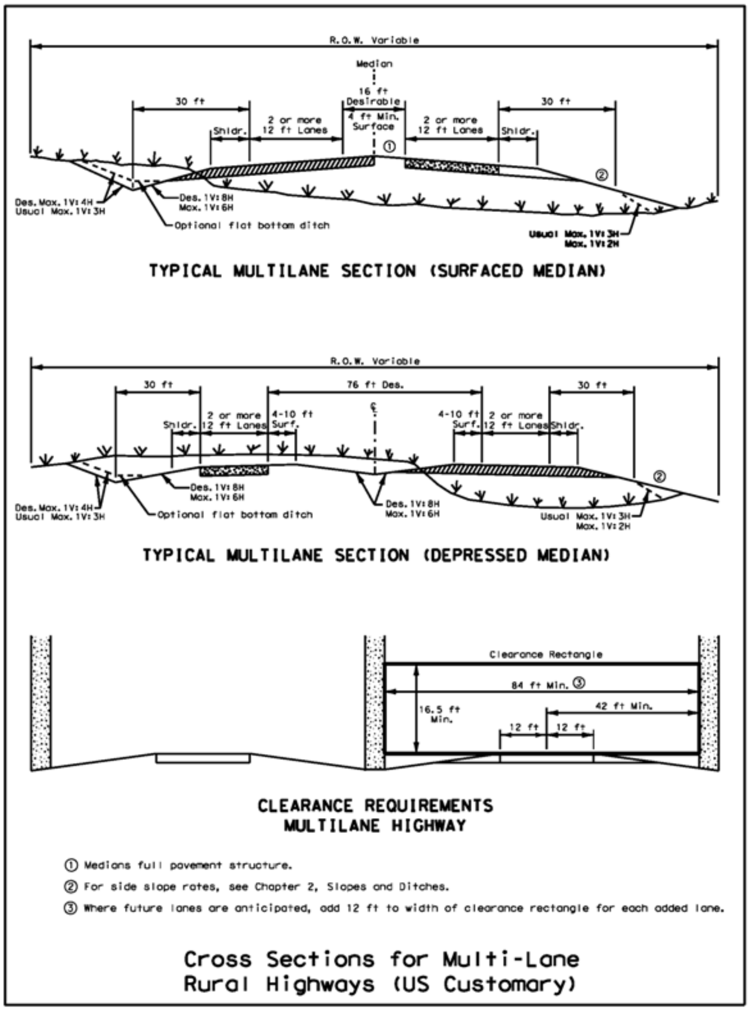


Figure 3-8. (US). Cross Sections For Multi-Lane Rural Highways. Click US Customary or Metric to see a PDF of the image.

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 2-3, Determination of Length of Superelevation Transition.
- Maximum Gradient: Table 2-9: Maximum Grades
- Fill Slope Rates: Table 2-10: 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 roadways have generally been associated with higher accident rates than divided roadways. This higher accident 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 intersections 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. Undivided highways may be constructed as betterment projects for existing two-lane highways to improve passing opportunities and traffic operations. Undivided highways are sometimes provided in 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 general 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 spacing 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 shopping 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 usually 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.

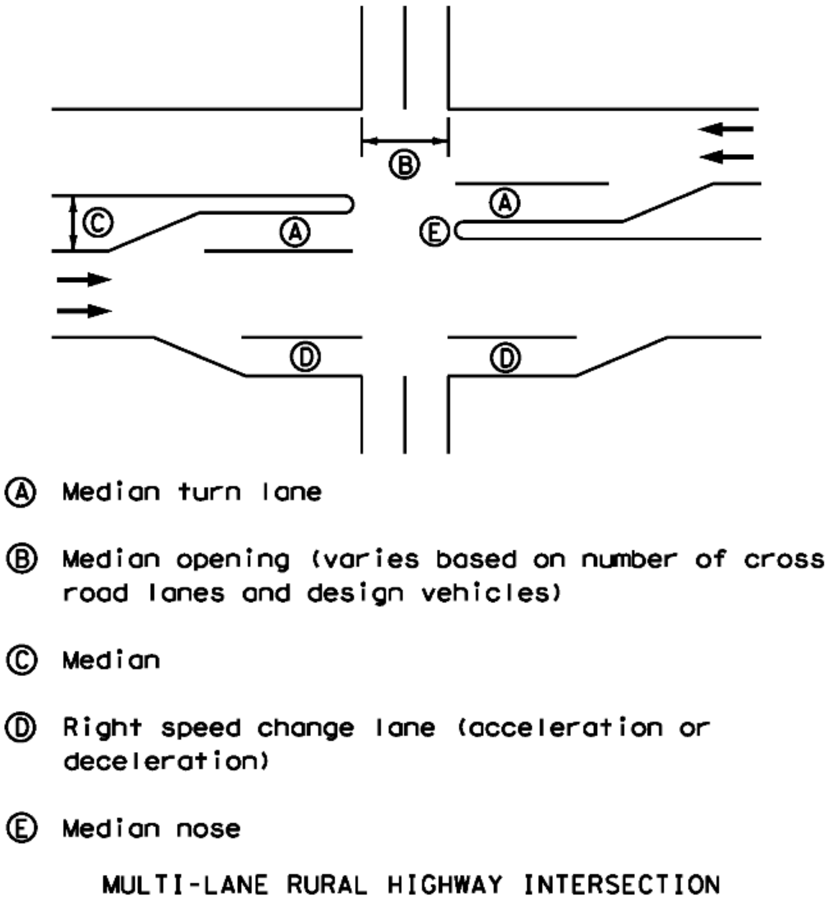


Figure 3-9. Multi-Lane Rural Highway Intersection. Click here to see a PDF of the image.

Median opening width should in no case be less than 40 ft [12 m] nor 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.

Turn Lanes

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

Median Turn Lane (Left-Turn Lane). Median turn lanes 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

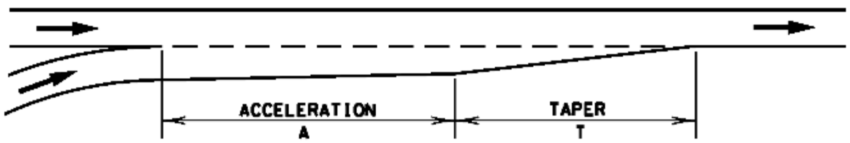
(US Customary)					
Mainlane Design Speed (mph)	Taper Length (ft)[1]	Deceleration Length (ft)[2]	-	Design Turning ADT (vpd)	Minimum Storage Length (ft)
30	50	160	-	150	50
35	50	215	-	300	100
40	50	275	-	500	175
45	100	345	-	750	250
50	100	425	-	--	--
55	100	510	-	--	--
60	150	615	-	--	--
65	150	715	-	--	--
70	150	830	-	--	--
75	150	950	-	--	--
80	150	1075	-	--	--
(Metric)					
Mainlane Design Speed (km/h)	Taper Length (m)[1]	Deceleration Length (m)[2]	-	Design Turning ADT (vpd)	Minimum Storage Length (m)
50	15	50	-	150	15
60	15	65	-	300	30
70	30	85	-	500	50
80	30	105	-	750	75
90	30	130	-	-	-
100	45	200	-	-	-
110	45	240	-	-	-
120	45	290	-	-	-
130	45	330	-	-	-

[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.



HIGHWAY DESIGN SPEED (mph)	MINIMUM LENGTH OF TAPER T (ft)	ACCELERATION LENGTH, A (ft) FOR ENTRANCE CURVE DESIGN SPEED (mph)								
		STOP CONDITION	15	20	25	30	35	40	45	50
		AND INITIAL SPEED (mph)								
		0	14	18	22	26	30	36	40	44
30	150	180	140	-	-	-	-	-	-	-
35	165	280	220	160	-	-	-	-	-	-
40	180	360	300	270	210	120	-	-	-	-
45	200	560	490	440	380	280	160	-	-	-
50	230	720	660	610	550	450	350	130	-	-
55	250	960	900	810	780	670	550	320	150	-
60	265	1200	1140	1100	1020	910	800	550	420	180
65	285	1410	1350	1310	1220	1120	1000	770	600	370
70	300	1620	1560	1520	1420	1350	1230	1000	820	580
75	330	1790	1730	1630	1580	1510	1420	1160	1040	780

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.

LENGTHS OF RIGHT-TURN ACCELERATION LANES
(US CUSTOMARY)

Figure 3-10. (US). Lengths of Right-Turn Acceleration Lanes. Click US Customary or Metric to see a PDF of the image.

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

(US Customary)				
Deceleration Lanes				
-	Ratio of Length on Grade to Length on Level*			
Design Speed of Roadway (mph)	3 to 4 % Upgrade	3 to 4 % Downgrade	5 to 6% Upgrade	5 to 6% Downgrade
All	0.9	1.2	0.8	1.35
-	Acceleration Lanes			
-	Ratio of Length on Grade to Length for Design Speed (mph) of Turning Roadway Curve*			

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

(US Customary)								
Design Speed of Roadway (mph)	20	25	30	35	40	45	50	All Speeds
-	3 to 4 % Upgrade							3 to 4% Downgrade
30	----	----	----	----	----	----	----	----
35	----	----	----	----	----	----	----	0.7
40	1.3	1.3	1.3	1.3	----	----	----	0.7
45	1.3	1.3	1.35	1.35	----	----	----	0.675
50	1.3	1.35	1.4	1.4	1.4	----	----	0.65
55	1.35	1.4	1.45	1.45	1.45	1.45	----	0.625
60	1.4	1.45	1.5	1.5	1.5	1.51.55	1.6	0.6
65	1.45	1.5	1.55	1.55	1.6	1..65	1.7	0.6
70	1.5	1.55	1.6	1.65	1.7	1.75	1.8	0.6
75	1.55	1.6	1.65	1.7	1.75	1.8	1.9	0.6
80	1.6	1.65	1.7	1.75	1.8	1.9	2.0	0.6
-	5 to 6% Upgrade							5 to 6% Downgrade
30	----	----	----	----	----	----	----	----
35	----	----	----	----	----	----	----	0.6
40	1.5	1.5	1.5	1.6	----	----	----	0.6
45	1.5	1.55	1.6	1.6	----	----	----	0.575
50	1.5	1.6	1.7	1.8	1.9	2.0	----	0.55
55	1.6	1.7	1.8	1.9	2.05	2.1	----	0.525
60	1.7	1.8	1.9	2.05	2.2	2.4	2.5	0.5
65	1.85	1.95	2.05	2.2	2.4	2.6	2.75	0.5
70	2.0	2.1	2.2	2.4	2.6	2.8	3.0	0.5
75	2.2	2.3	2.4	2.65	2.9	3.2	3.5	0.5
80	2.4	2.5	2.6	2.9	2.9	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.								

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

(Metric)				
Deceleration Lanes				
-	Ration of Length on Grade to Length on Level*			
Design Speed of Roadway (mph)	3 to 4 % Upgrade	3 to 4 % Downgrade	5 to 6% Upgrade	5 to 6% Downgrade

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

(Metric)						
All	0.9	1.2	0.8	1.35		
-	Acceleration Lanes					
-	Ratio of Length on Grade to Length for Design Speed (km/h of Turning Roadway Curve)*					
Design Speed of Roadway (km/h)	40	50	60	70	80	All Speeds
-	3 to 4 % Upgrade					3 to 4% Downgrade
50	----	----	----	----	----	----
60	1.3	1.4	1.4	----	----	0.7
70	1.3	1.4	1.4	1.5	----	0.65
80	1.4	1.5	1.5	1.5	1.6	0.65
90	1.4	1.5	1.5	1.5	1.6	0.6
100	1.5	1.6	1.7	1.7	1.8	0.6
110	1.5	1.6	1.7	1.7	1.8	0.6
120	1.5	1.6	1.7	1.7	1.8	0.6
130	1.5	1.6	1.7	1.7	1.8	0.6
-	5 to 6% Upgrade					5 to 6% Downgrade
50	----	----	----	----	----	----
60	1.5	1.5	----	----	----	0.6
70	1.5	1.6	1.7	----	----	0.6
80	1.5	1.7	1.9	1.8	----	0.55
90	1.6	1.8	2.0	2.1	2.2	0.55
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.						

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.

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 effect 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.

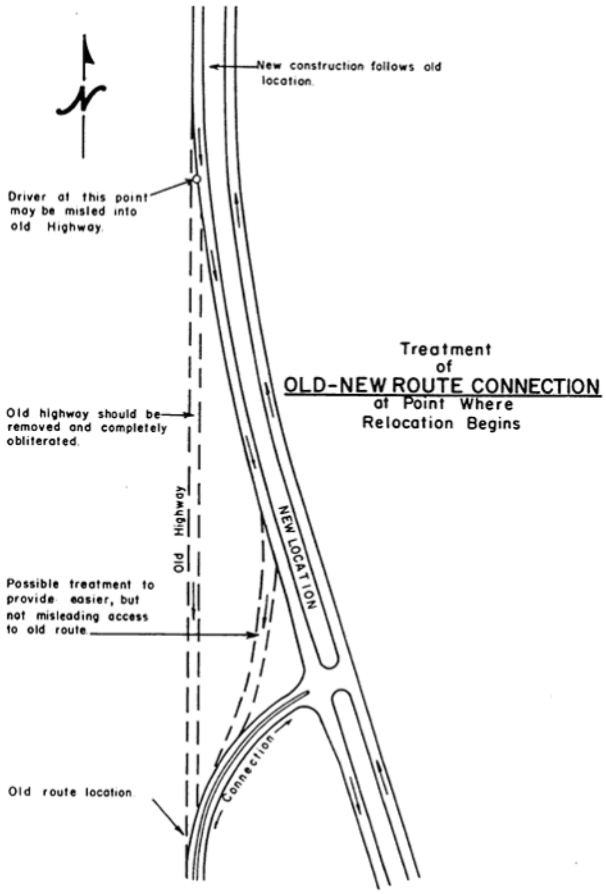


Figure 3-11. Treatment of Old-New Route Connection at Point Where Relocation Begins.

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.

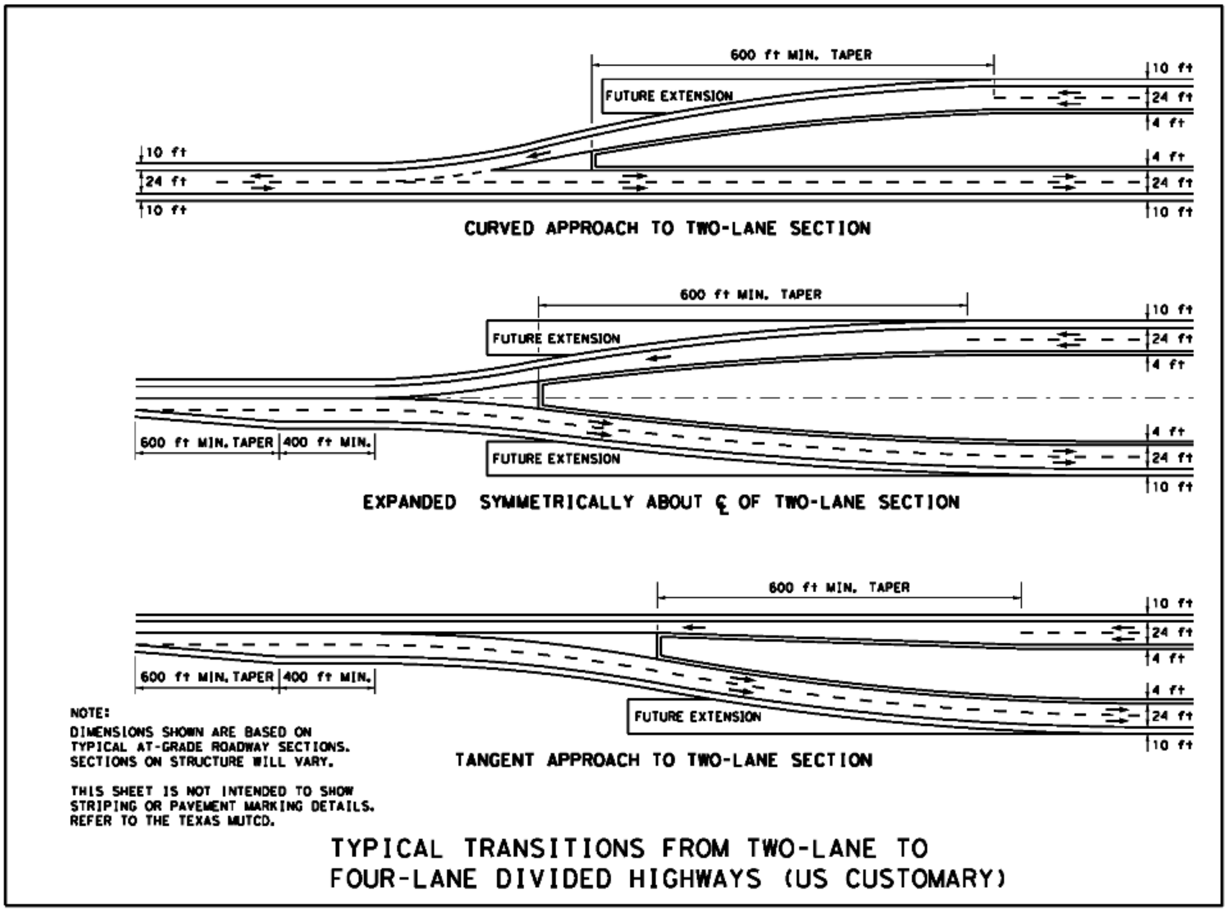


Figure 3-12. (US). Typical Transitions From Two-Lane To Four-Lane Divided Highways. Click US Customary or Metric to see a PDF of the image.

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.

An accident analysis of the existing two-lane roadway should be conducted. Any specific areas involving high accident 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 accident-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 Access
- Vertical and Horizontal Clearance at Structures
- Frontage Roads
- Interchanges

Basic Design Criteria

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

Table 3-15: Freeway Geometric Design Criteria

Design Criteria	Reference
General	
Horizontal Clearance	Table 2-11
Grades	Table 2-9
Minimum Horizontal Radius	Table 2-3 and 2-4
Superelevation	Tables 2-6, 2-7 and 2-8
Vertical Curvature	2-7, 2-8 ,2-9, and 2-10
Pavement Cross Slope	Chapter 2, Pavement Cross Slope
Freeways	
Design Speed Mainlanes (urban and rural)	Table 3-17
Design Speed Frontage Roads (urban and rural)	Chapter 3, Frontage Roads
Capacity and LOS Analysis	*Highway Capacity Manual*

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 a 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.

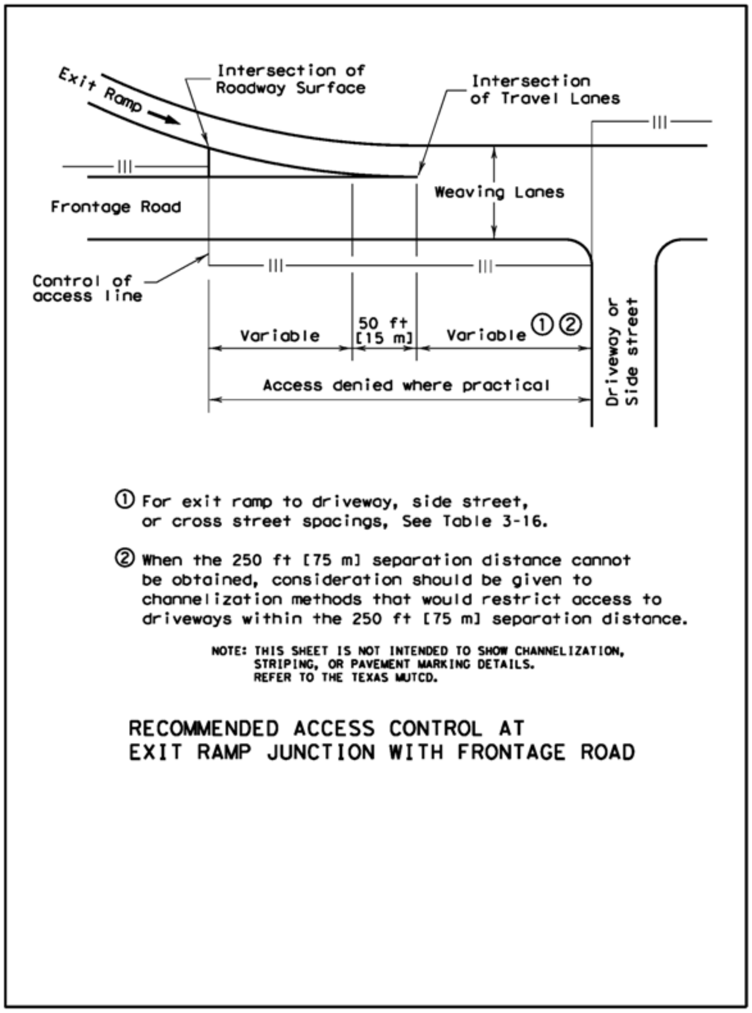


Figure 3-13. Recommended Access Control At Exit Ramp Junction With Frontage Road. Click here to see a PDF of the image.

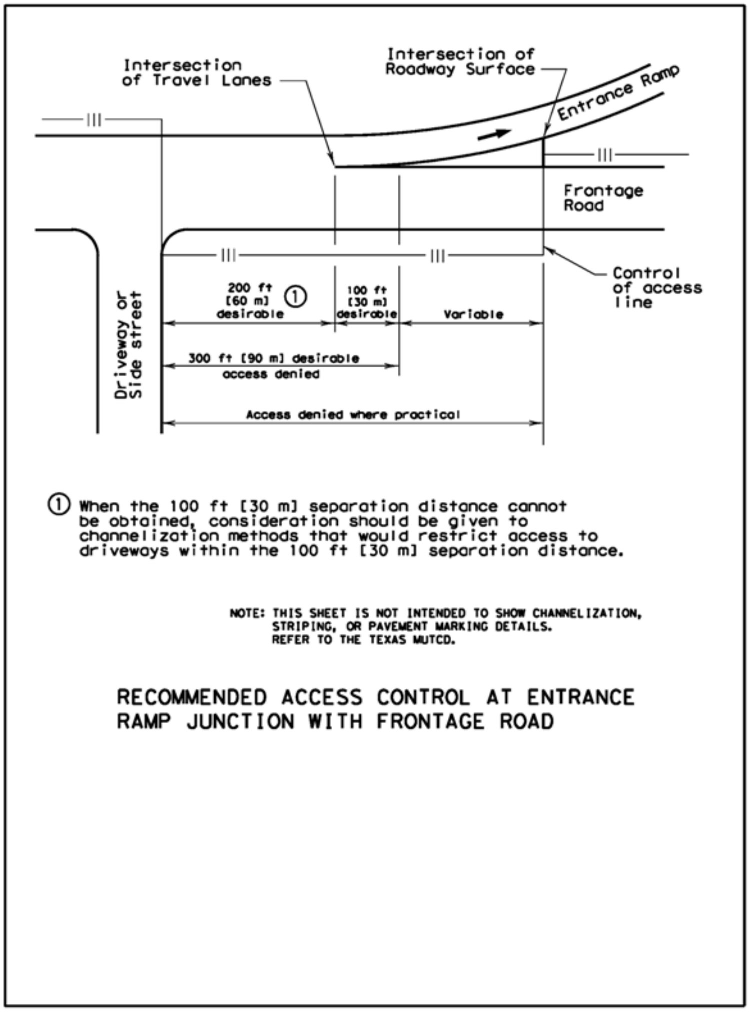


Figure 3-14. Recommended Access Control At Entrance Ramp Junction With Frontage Road. Click here to see a PDF of the image.

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.

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

Total Volume (Frtg rd +Ramp) (vph)	Driveway or Side Street Volume (vph)	Spacing (ft [m])		
--	--	Number of Weaving Lanes		
--	--	2	3	4
< 2500	< 250	460 [140]	460 [140]	560 [170]
--	> 250	520 [160]	460 [140]	560 [170]
--	> 750	790 [240]	460 [140]	560 [170]
--	> 1000	1000 [300]	460 [140]	560 [170]
> 2500	< 250	920 [280]	460 [140]	560 [170]
--	> 250	950 [290]	460 [140]	560 [170]
--	> 750	1000 [300]	600 [180]	690 [210]
--	> 1000	1000 [300]	1000 [300]	1000 [300]

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 may be necessary.

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. 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.

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)

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.

Design

If an existing highway is to be developed as a controlled access facility solely by design (not designated by the Transportation Commission), the Texas Department of Transportation 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.
- Landlocking of an abutting property does not result.

The Departmental publication entitled 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 if practical and feasible.

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.

Table 3-17 provides minimum design speeds for freeways:

Table 3-17: Design Speed for Controlled Access Facilities (mph [km/h])

Facility	Minimum
Mainlanes - Urban	50 [80]
Mainlanes - Rural	70 [110]

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 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 3-11 for further information.

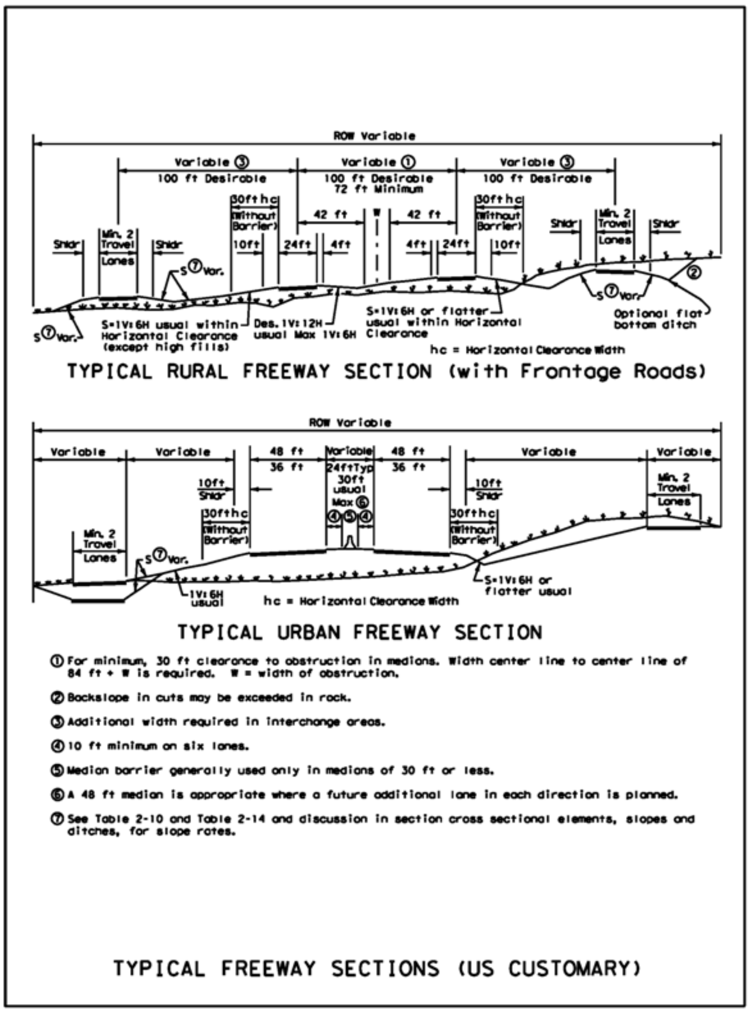


Figure 3-15. (US). Typical Freeway Sections. Click US Customary or Metric to see a PDF of the image.

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 3-11 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 3-11 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 Horizontal Clearance 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.

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. However, the above clearances should not generally be exceeded by more than approximately 3 inches [75 mm].

Vertical clearance for railroad overpasses is shown in Figure 3-16, and is discussed further in the *Bridge Project Development Manual*.

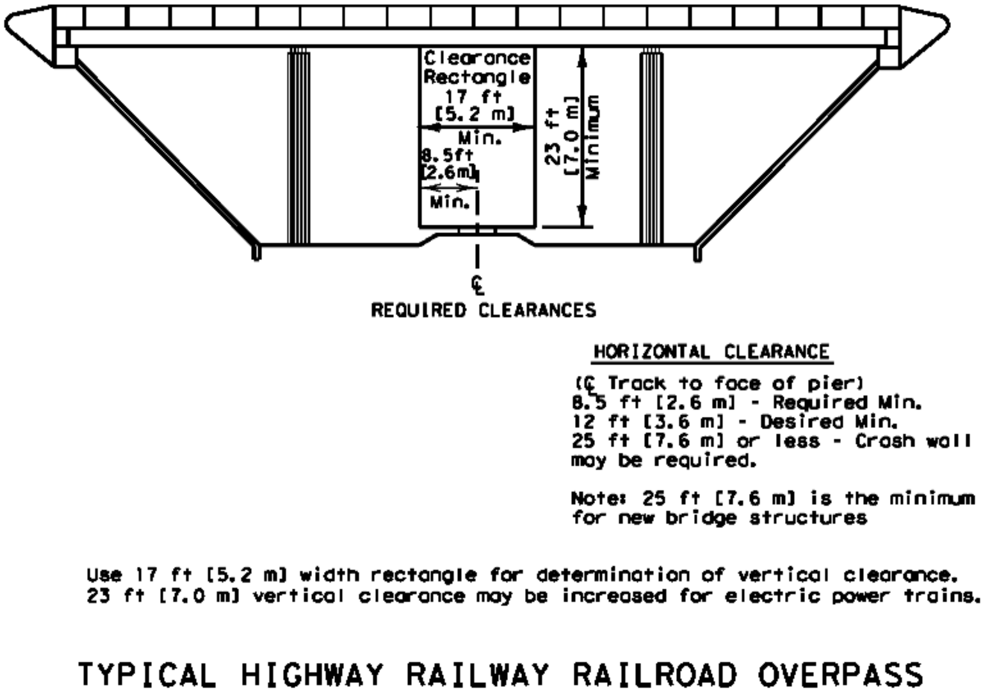


Figure 3-16. Typical Highway Railway Railroad Overpass. Click here to see a PDF of the image.

Horizontal. The minimum horizontal clearance to bridge parapets and piers should be as shown in Table 2-11: Horizontal Clearances and Figure 3-12.

Table 3-18: Roadway Widths for Controlled Access Facilities

(US Customary)				
Type of Roadway	Inside Shoulder Width (ft)	Outside Shoulder Width[2] (ft)		Traffic Lanes (ft)
Mainlanes:	-	-		-
4-Lane Divided	4	10		24
6-Lane or more Divided	10	10		361
1-Lane Direct Conn.2	2 Rdwy.; 4 Str.	8		14
2-Lane Direct Conn.	2 Rdwy.; 4 Str.	8		24
Ramps[2] (uncurbed)	2 Rdwy.; 4 Str.	Min.	Des.	14
-	-	6	8	-
Ramps[3] (curbed)	—	—		22
(Metric)				

Table 3-18: Roadway Widths for Controlled Access Facilities

(US Customary)				
Type of Roadway	Inside Shoulder Width (m)	Outside Shoulder Width[2] (m)		Traffic Lanes (m)
Mainlanes:	-	-		-
4-Lane Divided	1.2	3.0		7.2
6-Lane or more Divided	3.0	3.0		10.81
1-Lane Direct Conn.2	0.6 Rdwy.; 1.2 Str.	2.4		4.2
2-Lane Direct Conn.	0.6 Rdwy.; 1.2 Str.	2.4		7.2
Ramps[2] (uncurbed)	0.6 Rdwy.; 1.2 Str.	Min.	Des.	4.2
-	-	1.8	2.4	-
Ramps[3] (curbed)	—	—		6.6

[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 accidents 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
- 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 *Right of Way* Manual, Volume 1, 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
- 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. Horizontal clearances are given in Table 2-11: Horizontal Clearances.

Any frontage road constructed will be designed to provide one-way operation initially. 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

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

Table 3-19: Design Criteria for Rural Frontage Roads

(US Customary)				
45	10	11	11	12
50	10	11	12	12
55	10	11	12	12
60	11	11	12	12
65	11	11	12	12
70	11	11	12	12
75	11	12	12	12
80	11	12	12	12
SHOULDERS (ft)4				
Each Shoulder Two-Way Operation	2[3]	4	8	8 – 10
Inside Shoulder One-Way Operation	2[3]	2[3]	4	4[4]
Outside Shoulder One-Way Operation	2[3]	4	8	8 – 10

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

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

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

[4]If the one-way frontage road section contains three or more travel lanes, then minimum inside shoulder width is 8 – 10 ft.

Table 3-19: Design Criteria for Rural Frontage Roads

(Metric)				
Design Speed[2] (km/h)	Min. Width[1] for Future Traffic Volume of			
-	0-400 ADT	400-1,500 ADT	1,500-2,000 ADT	2,000 or more ADT
LANES (m)				
30	3.0	3.0	3.3	3.6
40	3.0	3.0	3.3	3.6
50	3.0	3.0	3.3	3.6
60	3.0	3.3	3.3	3.6
70	3.0	3.3	3.3	3.6
80	3.0	3.3	3.6	3.6
90	3.0	3.3	3.6	3.6

Table 3-19: Design Criteria for Rural Frontage Roads

(Metric)				
100	3.3	3.3	3.6	3.6
110	3.3	3.3	3.6	3.6
120	3.3	3.6	3.6	3.6
130	3.3	3.6	3.6	3.6
SHOULDERS (m)4				
Each Shoulder Two-Way Operation	0.6[3]	1.2	2.4	2.4 – 3.0
Inside Shoulder One-Way Operation	0.6[3]	0.6[3]	1.2	1.2[4]
Outside Shoulder One-Way Operation	0.6[3]	1.2	2.4	2.4 – 3.0

[1]May retain existing paved width on a reconstruction project if total paved width is 7.2 m and operating satisfactorily.
[2]Use rural collector criteria (Table 3-6) for determining minimum design speed.
[3]At locations where roadside barriers are provided, use minimum 1.2 m offset from travel lane edge to barrier face.
[4]If the one-way frontage road section contains three or more travel lanes, then minimum inside shoulder width is 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.
- Accident rate comparisons are above the statewide average accident 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
- Four Leg Interchanges

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 3-17. 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.

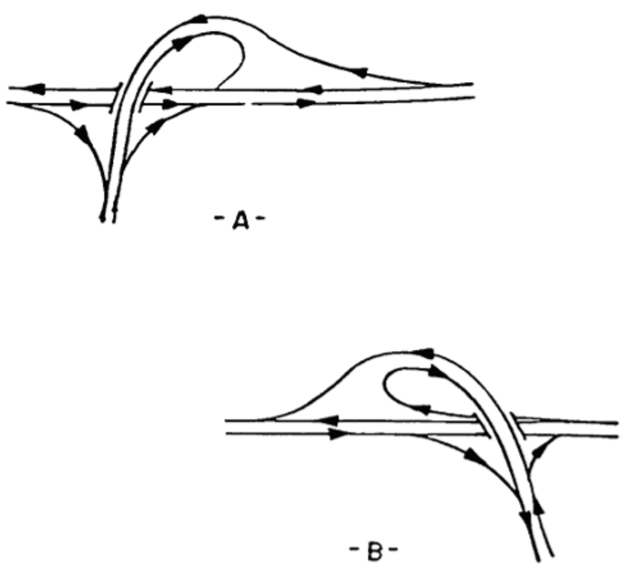


Figure 3-17. Trumpet Three Leg Interchange.

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 3-18.

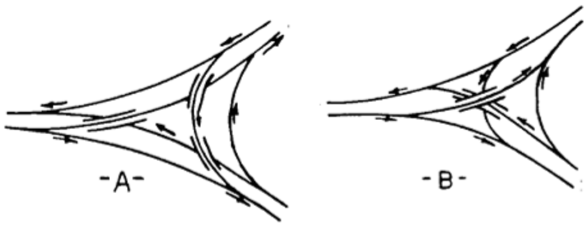


Figure 3-18. Directional Three Leg Interchange.

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
- 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 3-19.

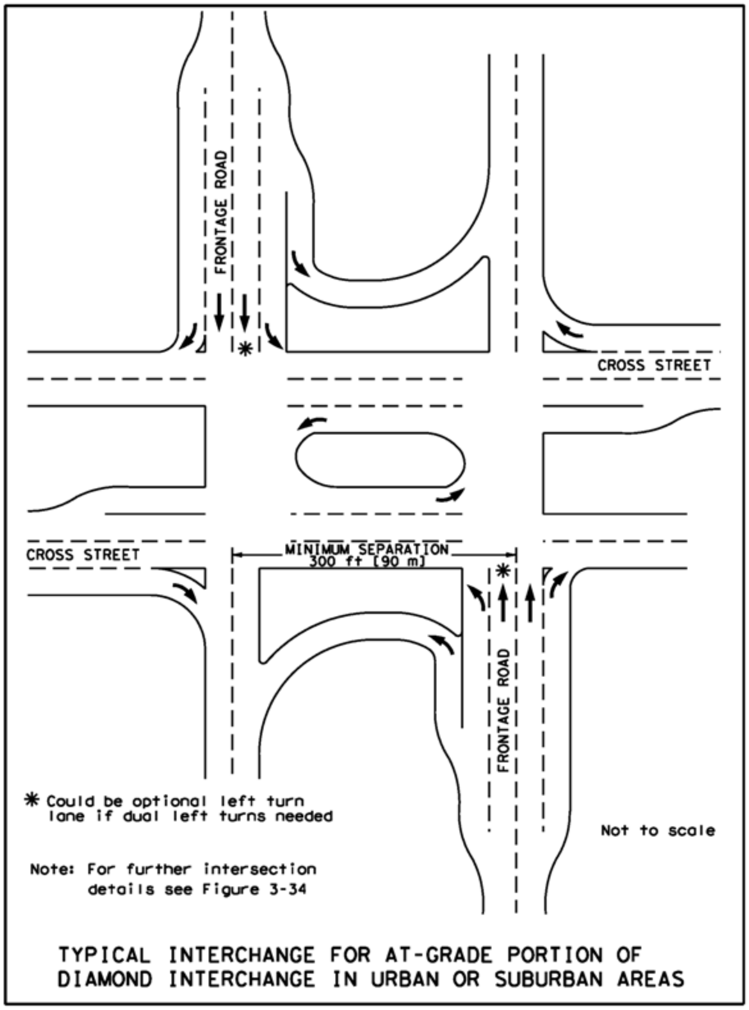


Figure 3-19. Typical Interchange For At-Grade Portion Of Diamond Interchange In Urban Or Suburban Areas. Click here to see a PDF of the image.

The diamond interchange may have several different configurations, as discussed in the following paragraphs and shown in Figure 3-20:

Conventional diamond without frontage roads. The conventional diamond (Figure 3-20 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 3-20 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 3-20 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 3-20 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 3-20 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 3-20 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.

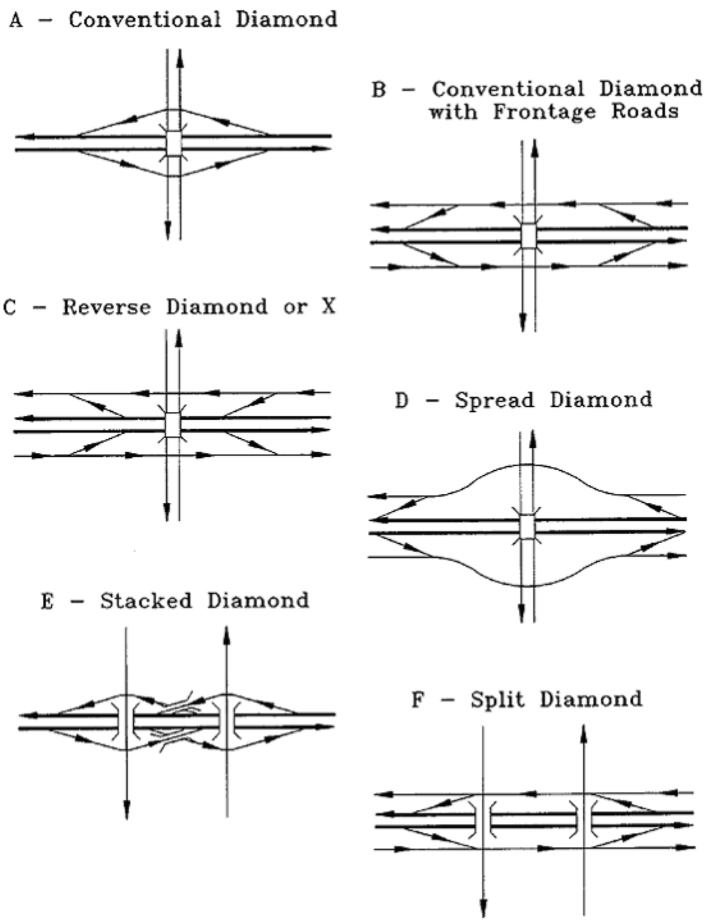


Figure 3-20. Typical Diamond Interchanges.

Three level diamond. In urban areas, where the cross street carries a high volume of traffic, the three-level diamond interchange, illustrated in Figure 3-21, 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.

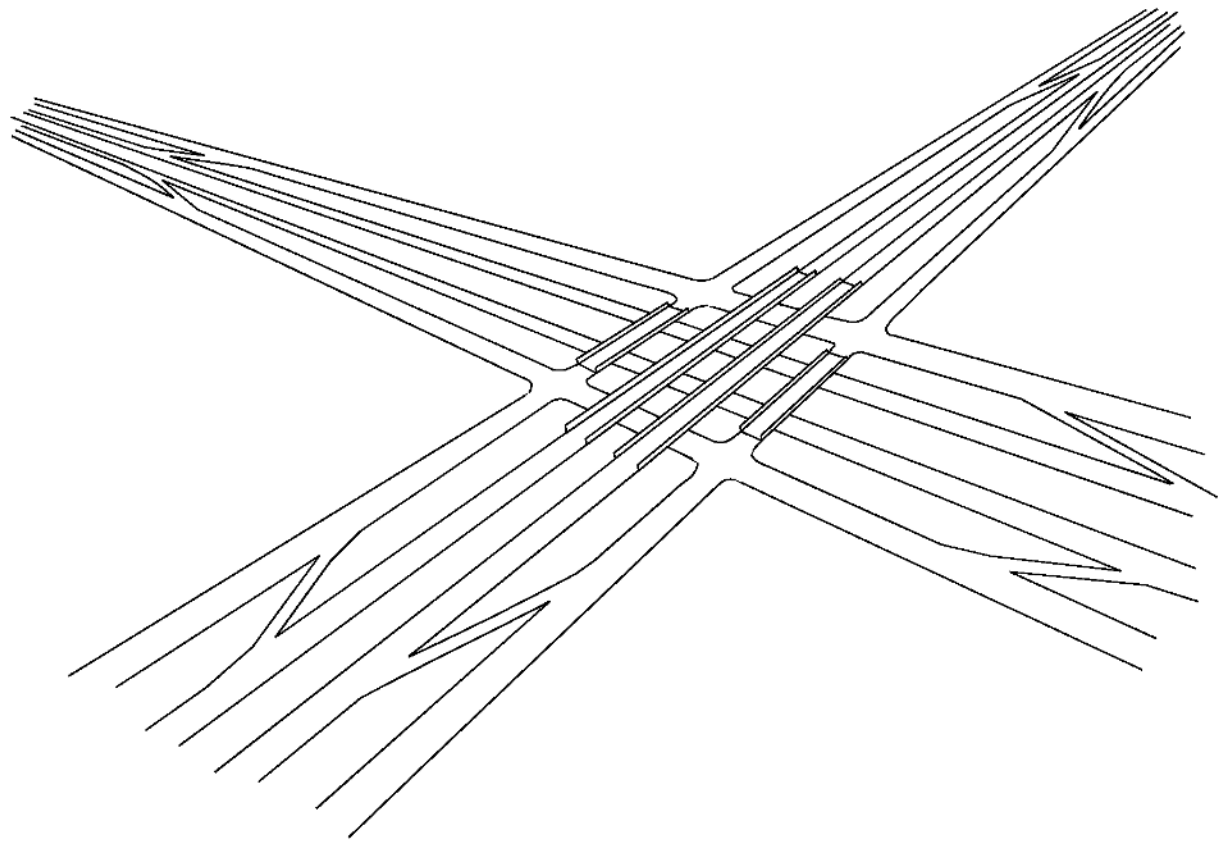


Figure 3-21. Three Level Diamond Interchange.

Single point diamond. A special type of freeway-to-arterial interchange has received attention during recent years and is worthy of discussion. AASHTO's *A Policy on Geometric Design of Highways and Streets* refers to it as a "single point diamond" 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 3-22. This type of interchange has application only in specialized locations. Traffic operations and signalization must be carefully modeled prior to final design selection of the single point urban interchange.

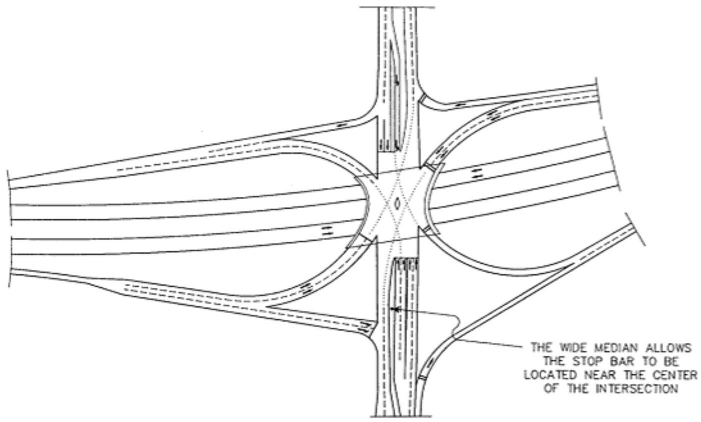


Figure 3-22. Single Point Diamond Interchange.

Three level stacked diamond. The three-level stacked diamond interchange is also an interchange requiring only one signalized intersection. In a sense, it is a three-level version of the "single point diamond" configuration, as illustrated in Figure 3-23. This design grade separates both roadways, and accommodates turning movements with signal operations requiring only one signalized intersection. The two-phase signal operation at the intersection typically provides a level of throughput on the turning movements between a conventional diamond interchange and a fully directional interchange. Furthermore, it works best at separating high arterial cross-street and freeway traffic. It has the same shortcomings as the "single point diamond" in the way it brings the left turn movements together.

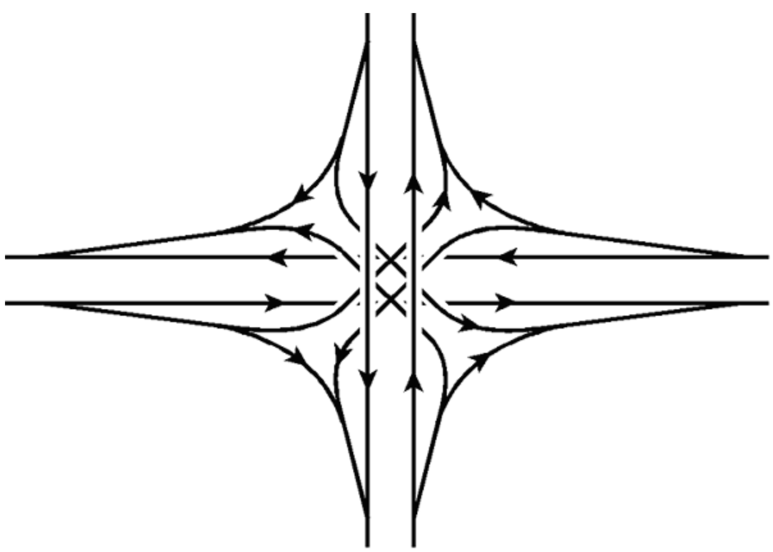


Figure 3-23. Three Level Stacked Diamond Interchange (see Figure 3-24 for At-Grade Portions of the Interchange).

As indicated in Figure 3-24, vehicles enter the intersection with oncoming vehicles to the right in contrast to the left as is the case on conventional diamond interchange intersections. Also, the design is less attractive with continuous frontage roads.

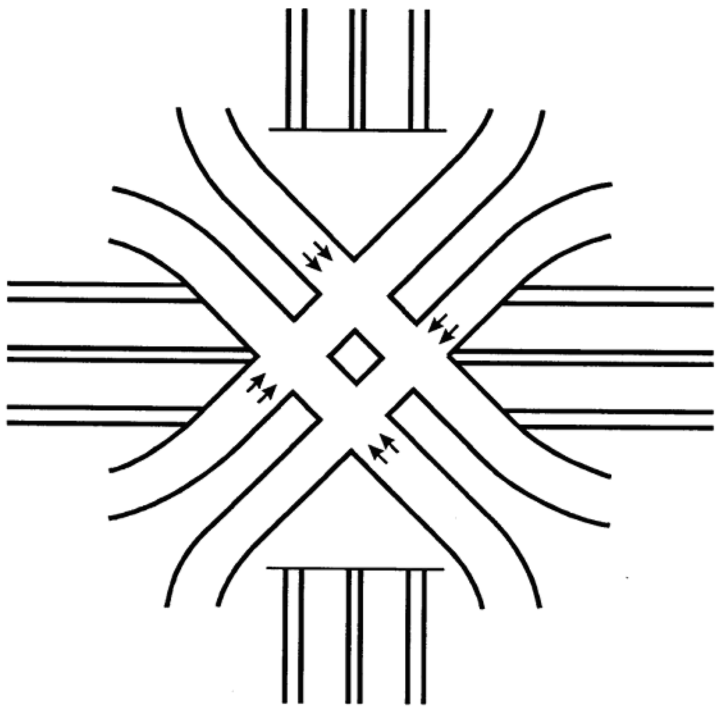


Figure 3-24. Three Level Stacked Diamond At-Grade Interchange.

Cloverleaf Interchanges

Cloverleaf interchanges are very common in many states. These types of interchanges were popular in the early era of freeway construction, but are usually no longer considered preferable for freeway to freeway movement, especially when interchange volumes are high. However, in some instances they may be appropriate when 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 pcph) since loop ramps are limited to one lane of operation and have restricted operating speeds.

Primary disadvantages of the cloverleaf design include the following:

- large right-of-way requirements
- capacity restrictions of loops, especially if truck volumes are significant
- short weaving length between loops
- 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 3-25, eliminates all left-turn conflicts through construction of a two-level interchange.

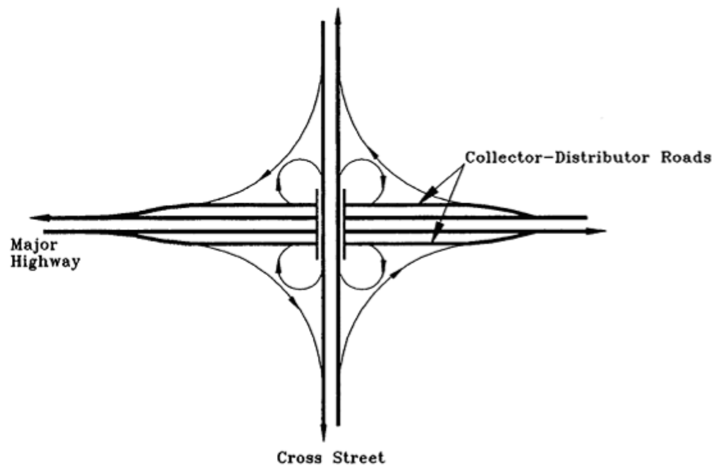


Figure 3-25. Full Cloverleaf Interchange.

Partial cloverleaf. A cloverleaf without ramps in all four quadrants, illustrated in Figure 3-26, 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.**

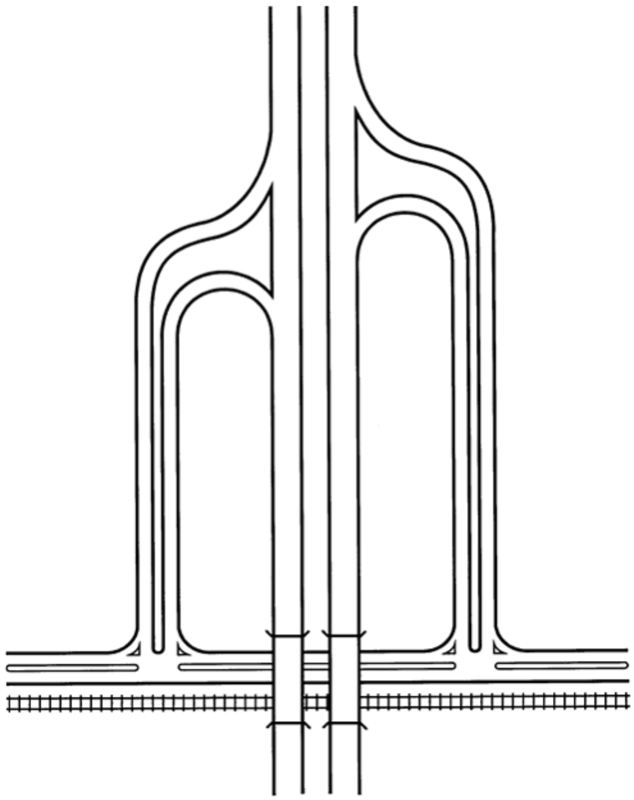


Figure 3-26. Partial Cloverleaf Interchange.

Directional Interchanges

Interchanges that use direct or semi-direct connections for one or more left-turn movements are called “directional” interchanges (Figure 3-27). 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.

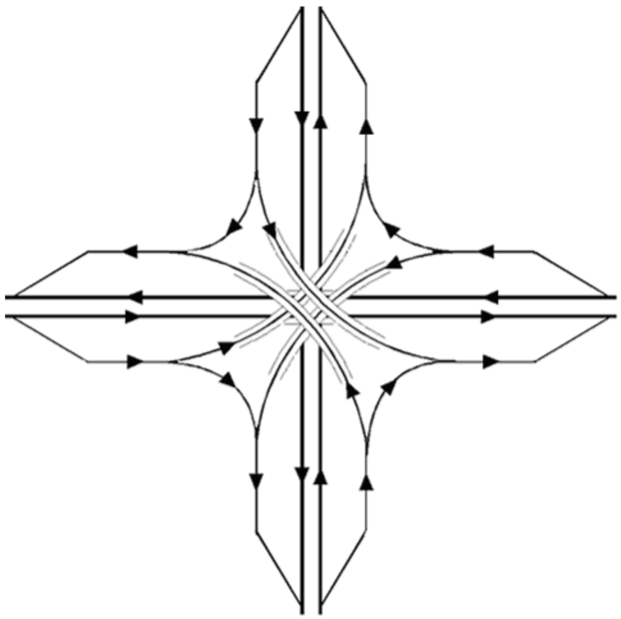


Figure 3-27. Four Level Fully Directional Interchange Without Frontage Roads.

Four level without frontage roads. The four-level directional interchange as depicted in Figure 3-27 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 3-28 depicts a five level interchange with frontage roads.

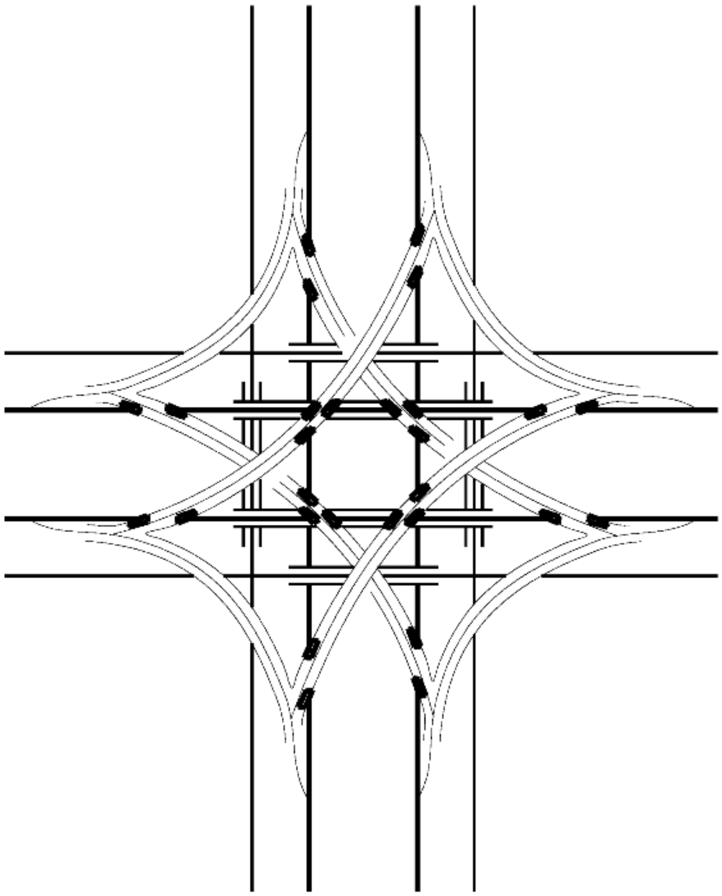


Figure 3-28. Five Level Fully Directional Interchange with Frontage Roads.

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
- 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 3-29 through 3-35.

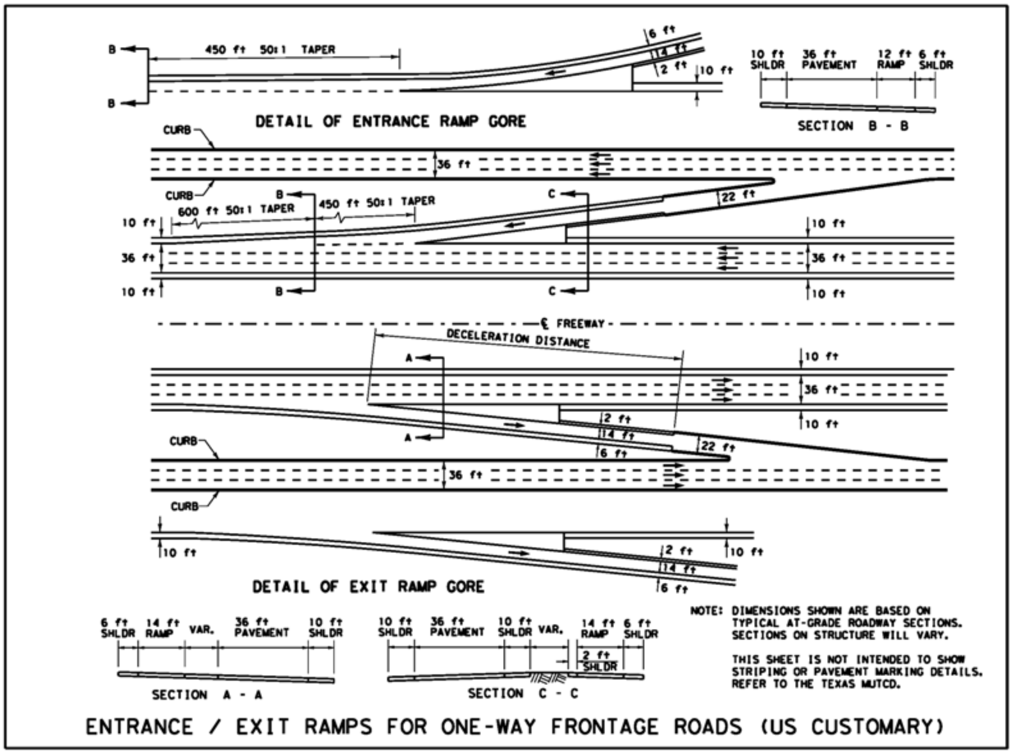


Figure 3-29. (US). Entrance/Exit Ramps For One-Way Frontage Roads. Click US Customary or Metric to see a PDF of the image.

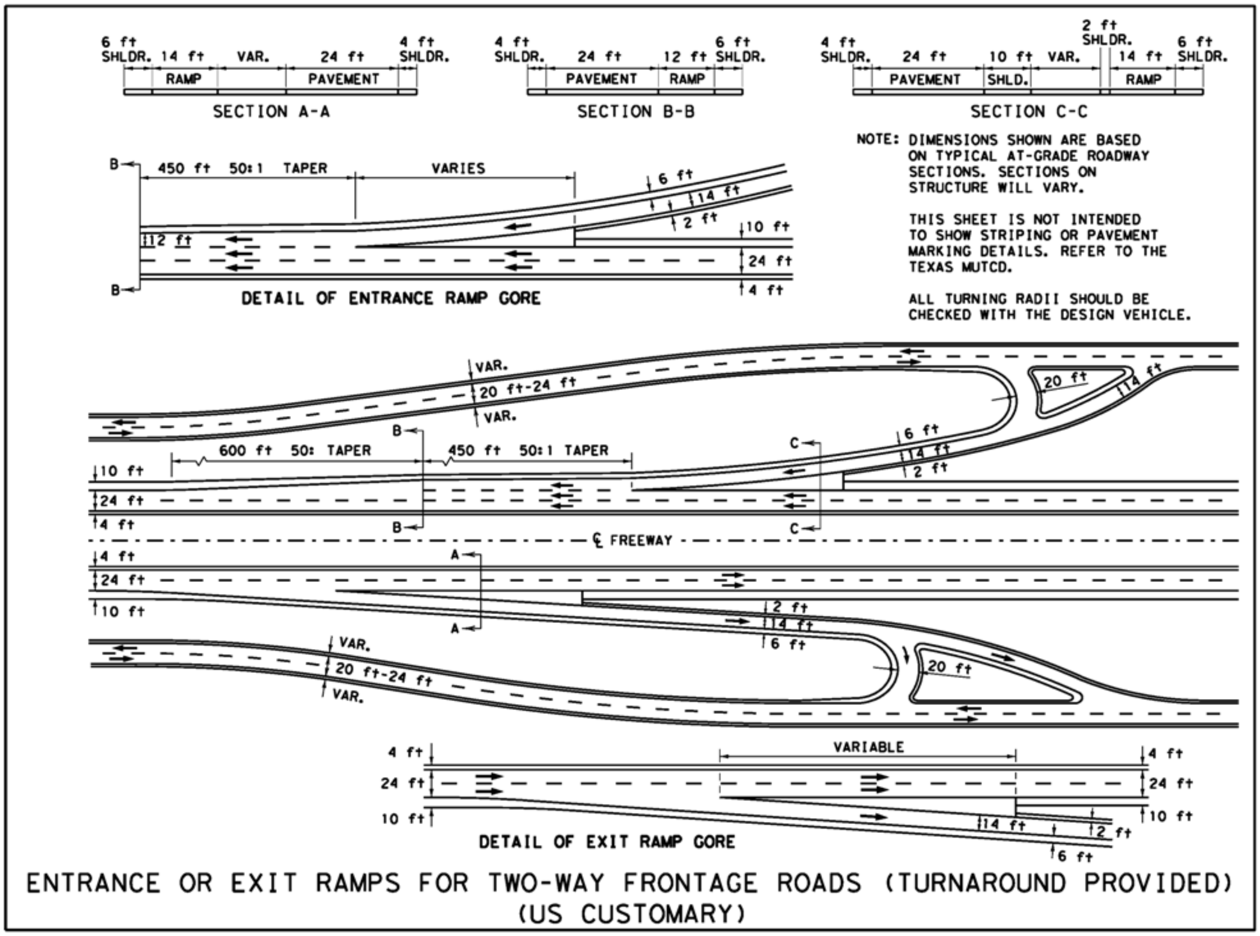


Figure 3-30. (US). Entrance Or Exit Ramps For Two-Way Frontage Roads (Turnaround Provided). Click US Customary or Metric to see a PDF of the image.

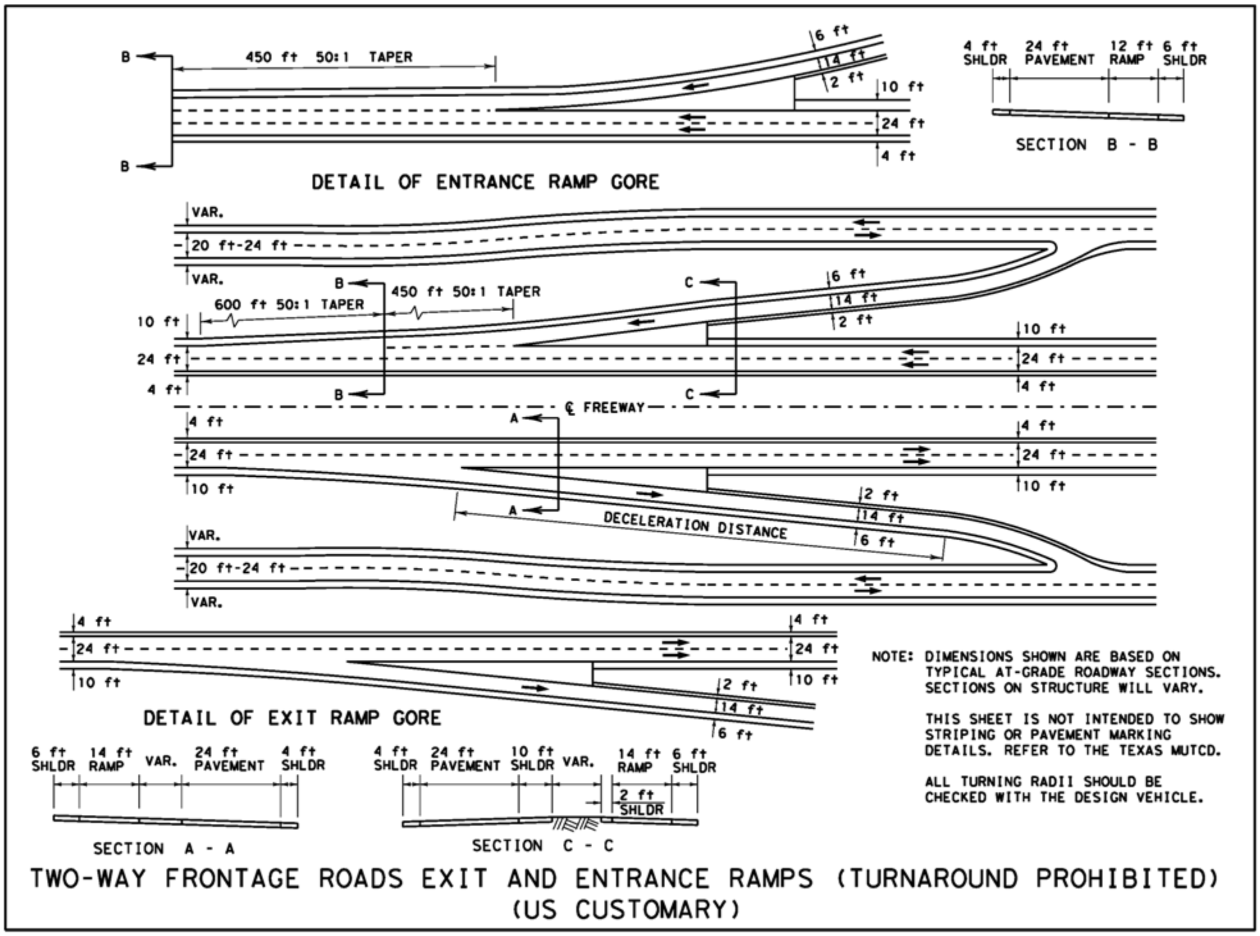


Figure 3-31. (US). Two-Way Frontage Roads Exit and Entrance Ramps (Turnaround Prohibited). Click US Customary or Metric to see a PDF of the image.

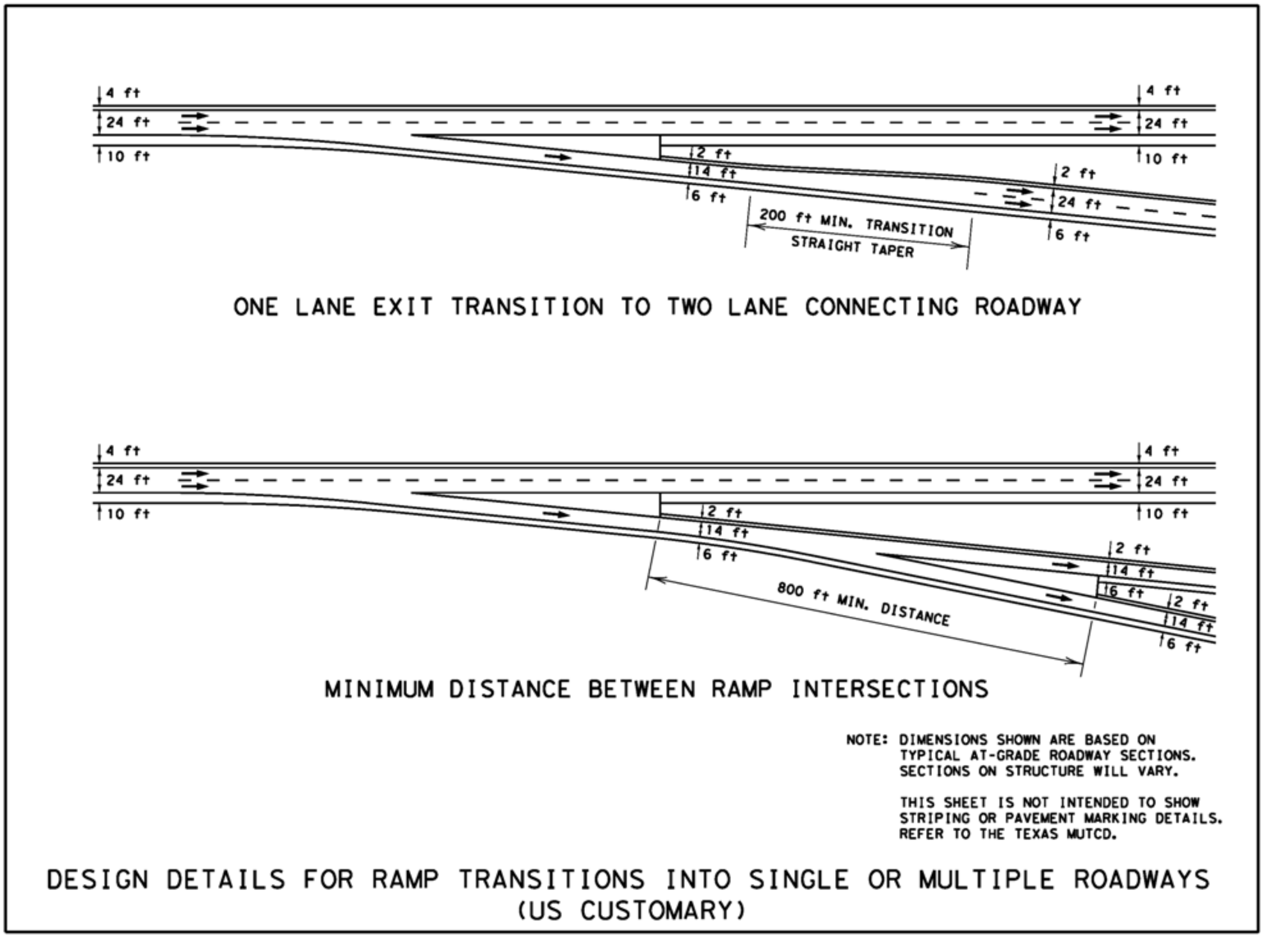


Figure 3-32. (US). Design Details For Ramp Transitions Into Single or Multiple Roadways. Click US Customary or Metric to see a PDF of the image.

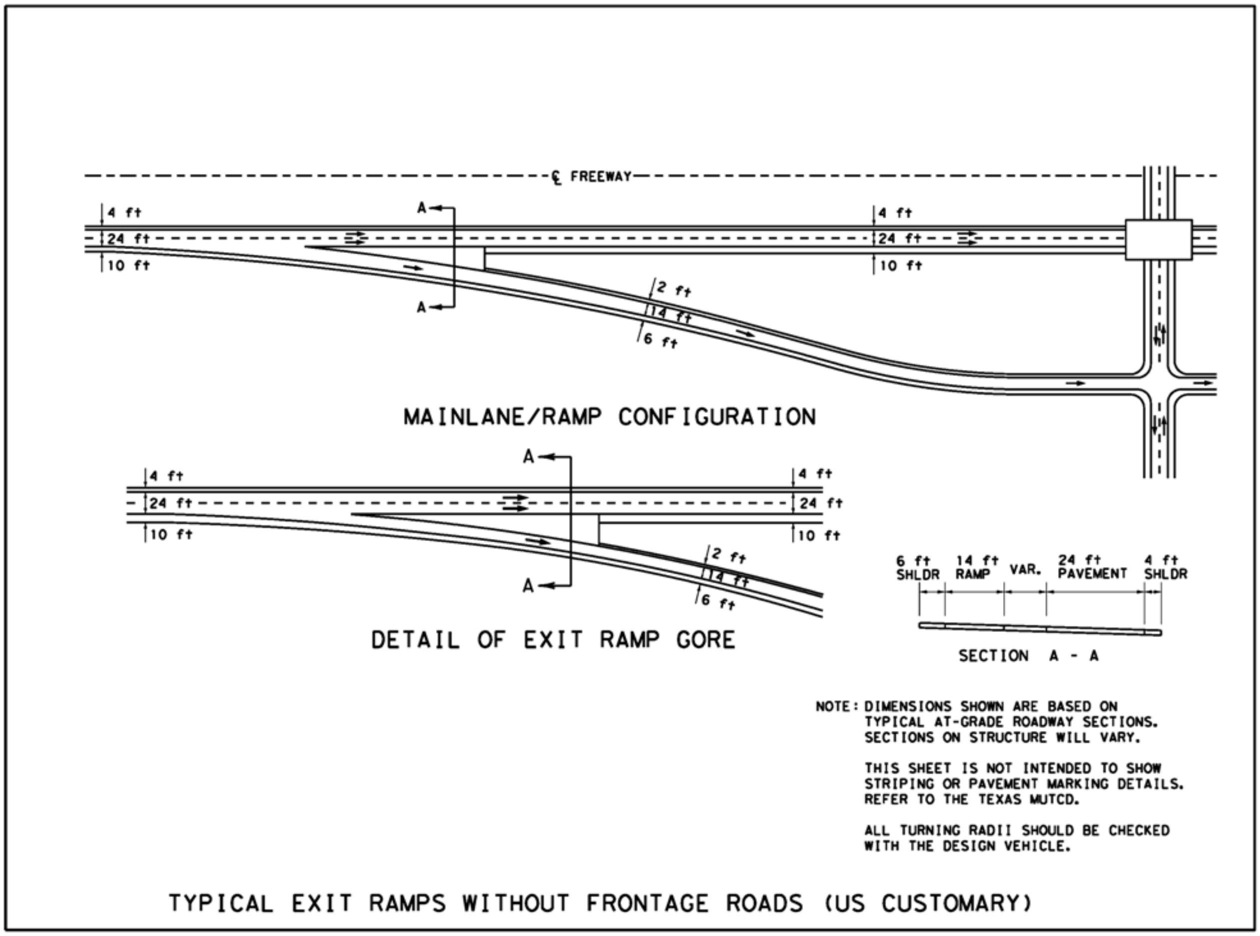


Figure 3-33. (US). Typical Exit Ramps Without Frontage Roads. Click US Customary or Metric to see a PDF of the image.

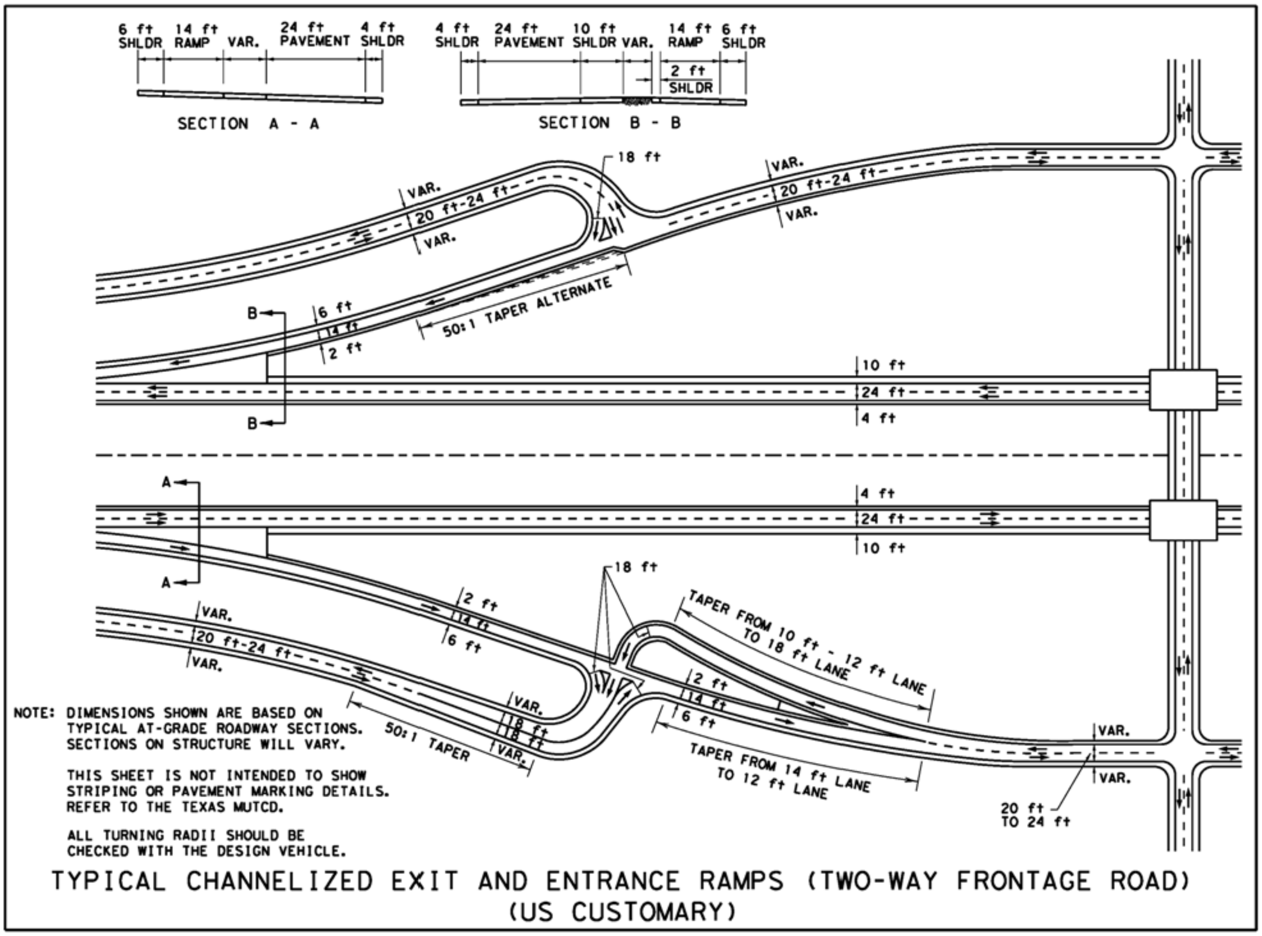


Figure 3-34. (US). Typical Channelized Exit and Entrance Ramps (Two-Way Frontage Road). Click US Customary or Metric to see a PDF of the image.

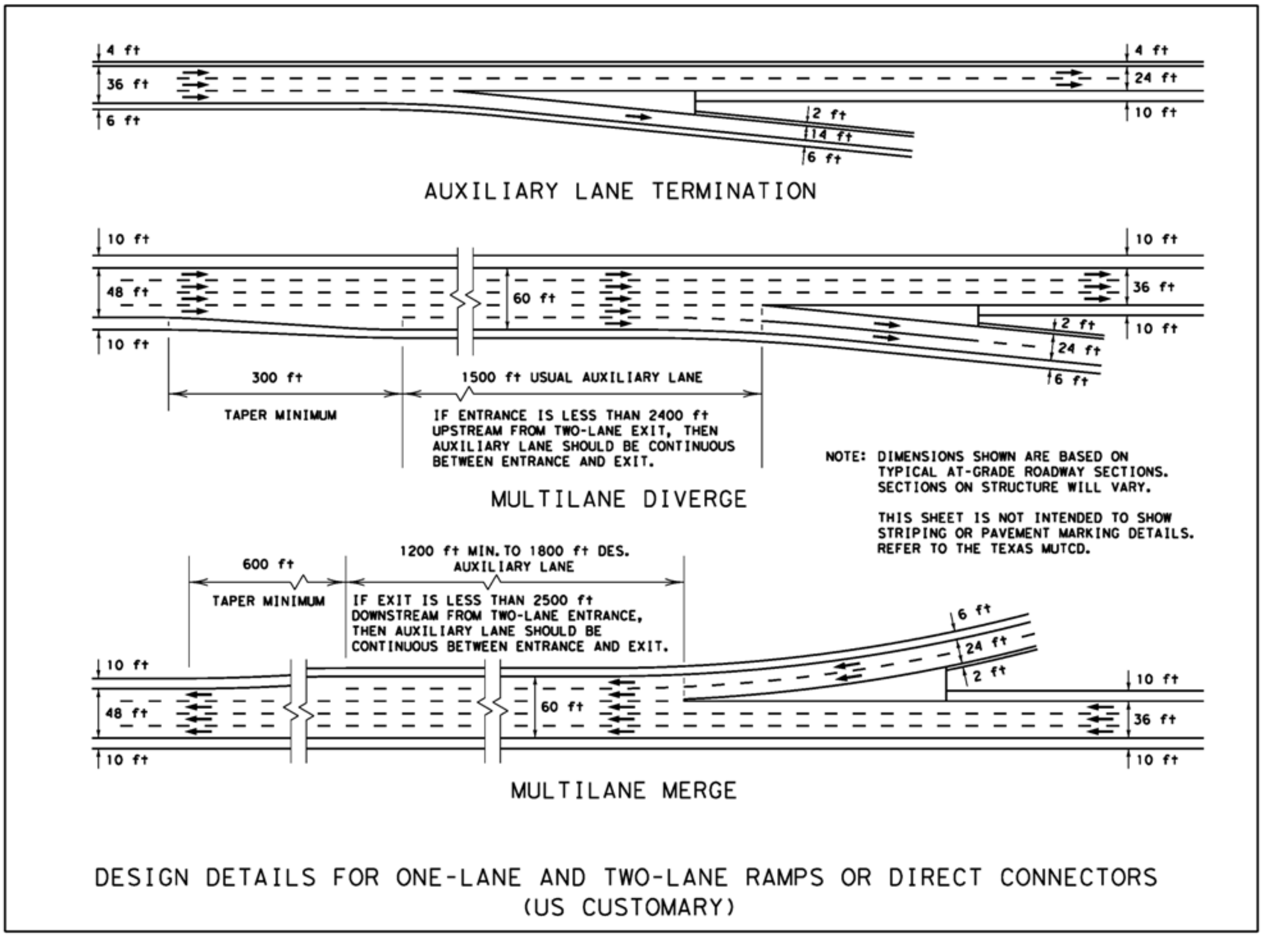


Figure 3-35. (US). Design Details for One-Lane and Two-Lane Ramps or Direct Connectors. Click US Customary or Metric to see a PDF of the image.

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. 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*

(US Customary)											
Highway Design Speed (mph)	30	35	40	45	50	55	60	65	70	75	80
Ramp** Design Speed (mph):	-										
Upper Range (85%)	25	30	35	40	45	48	50	55	60	65	70
Mid Range (70%)	20	25	30	33	35	40	45	45	50	55	60
Lower Range (50%)	15	18	20	23	25	28	30	30	35	40	45
(Metric)											
Highway Design Speed (km/h)	50	60	70	80	90	100	110	120	130		
Ramp** Design Speed (km/h):	-										
Upper Range (85%)	40	50	60	70	80	90	100	110	120		
Mid Range (70%)	30	40	50	60	60	70	80	90	100		
Lower Range (50%)	20	30	40	40	50	50	60	70	80		

* 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] (185 ft [55 m] minimum radius) based on an e_{max} of 6%. Particular attention should be given to controlling superelevation on loops due to the tight turning radii and speed limitations.

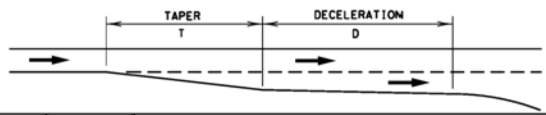
Horizontal Geometrics

Lane and shoulder widths for ramps and direct connections are shown in Table 3-18.

Figure 3-36 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

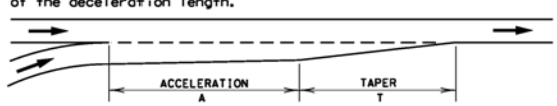
Exit and entrance ramp typical details are shown in Figure 3-23, Figure 3-24, Figure 3-25, Figure 3-26, Figure 3-27, Figure 3-28, and Figure 3-29.

Channelized (braided) entrance and exit ramps, as typified in Figure 3-28, 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.



HIGHWAY DESIGN SPEED (mph)	MINIMUM LENGTH OF TAPER T (ft)	DECELERATION LENGTH, D (ft) FOR ENTRANCE CURVE DESIGN SPEED (mph)								
		STOP CONDITION	15	20	25	30	35	40	45	50
		AND INITIAL SPEED (mph)								
		0	14	18	22	26	30	36	40	44
30	150	235	200	170	140	-	-	-	-	-
35	165	280	250	210	185	150	-	-	-	-
40	180	320	295	265	235	185	155	-	-	-
45	200	385	350	325	295	250	220	-	-	-
50	230	435	405	385	355	315	285	225	175	-
55	250	480	455	440	410	380	350	285	235	-
60	265	530	500	480	460	430	405	350	300	240
65	285	570	540	520	500	470	440	390	340	280
70	300	615	590	570	550	520	490	440	390	340
75	330	660	635	620	600	575	535	490	440	390

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.



HIGHWAY DESIGN SPEED (mph)	MINIMUM LENGTH OF TAPER T (ft)	ACCELERATION LENGTH, A (ft) FOR ENTRANCE CURVE DESIGN SPEED (mph)								
		STOP CONDITION	15	20	25	30	35	40	45	50
		AND INITIAL SPEED (mph)								
		0	14	18	22	26	30	36	40	44
30	150	180	140	-	-	-	-	-	-	-
35	165	280	220	160	-	-	-	-	-	-
40	180	360	300	270	210	120	-	-	-	-
45	200	560	490	440	380	280	160	-	-	-
50	230	720	660	610	550	450	350	130	-	-
55	250	960	900	810	780	670	550	320	150	-
60	265	1200	1140	1100	1020	910	800	550	420	180
65	285	1410	1350	1310	1220	1120	1000	770	600	370
70	300	1620	1560	1520	1420	1350	1230	1000	820	580
75	330	1790	1730	1630	1580	1510	1420	1160	1040	780

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.

LENGTHS OF RIGHT-TURN SPEED CHANGE LANES
(US CUSTOMARY)

Figure 3-36. (US). Lengths of Exit and Entrance Ramp Speed Change Lanes. Click US Customary or Metric to see a PDF of the image.

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 3-37 shows minimum distances between ramps for various ramp configurations.

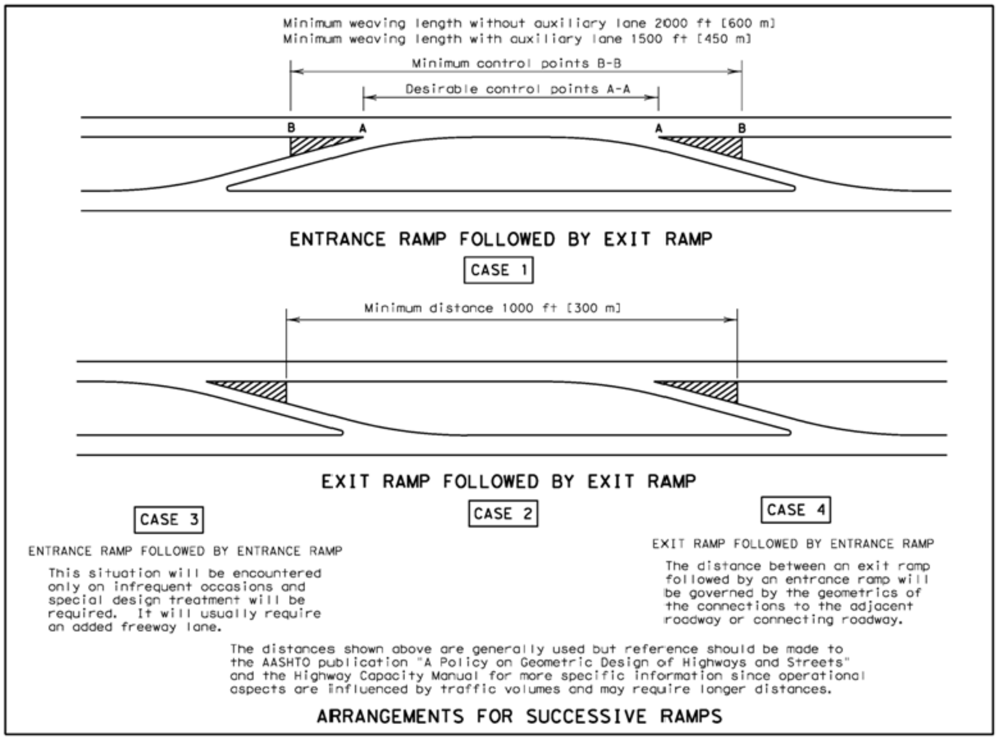


Figure 3-37. Arrangements For Successive Ramps. Click here to see a PDF of the image.

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.

Table 3-21: Superelevation Range for Curves on Connecting Roadways

(US Customary)						
Radius (ft)	Range in Superelevation Rate (percent) For Connecting Roadways With Design Speed, mph, of:					
-	20	25	30	35	40	45
90	2-10	-	-	-	-	-
150	2-8	4-10	-	-	-	-
230	2-6	3-8	6-10	-	-	-

Table 3-21: Superelevation Range for Curves on Connecting Roadways

(US Customary)						
310	2-5	3-6	5-9	8-10	-	-
430	2-4	3-5	4-7	6-9	9-10	-
540	2-3	3-5	4-6	6-8	8-10	10-10
600	2-3	2-4	3-5	5-7	7-9	8-10
1000	2	2-3	3-4	4-5	5-6	7-9
1500	2	2	2-3	3-4	4-5	5-6
2000	2	2	2	2-3	3-4	4-5
3000	2	2	2	2	2-3	3-4
* See Tables 2-6 and 2-7 for design speeds greater than 45 mph.						
(Metric)						
Radius (m)	Range in Superelevation Rate (percent) For Connecting Roadways With Design Speed, km/h, of:					
-	20	30	40	50	60	70
15	2-10	-	-	-	-	-
25	2-9	2-10	-	-	-	-
50	2-8	2-8	4-10	-	-	-
80	2-6	2-6	3-8	6-10	-	-
100	2-5	2-4	3-6	5-8	-	-
115	2-3	2-4	3-6	5-8	8-10	-
150	2-3	2-3	3-5	4-7	6-8	-
160	2-3	2-3	3-5	4-7	6-8	10-10
200	2	2-3	2-4	3-5	5-7	6-8
300	2	2-3	2-3	3-4	4-5	5-7
500	2	2	2	2-3	3-4	4-5
700	2	2	2	2	2-3	3-4
1000	2	2	2	2	2	2-3
* See Tables 2-6 and 2-7 for design speeds greater than 70 km/h.						

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%.

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

(US Customary)		(Metric)	
Design Speed of Exit or Entrance Curve (mph)	Maximum Algebraic Difference in Cross Slope at Crossover Line (%)	Design Speed of Exit or Entrance Curve (km/h)	Maximum Algebraic Difference in Cross Slope at Crossover Line (%)
Less than or equal to 20	5.0 to 8.0	Less than or equal to 30	5.0 to 8.0
25 to 30	5.0 to 6.0	40 to 50	5.0 to 6.0
Greater than or equal to 35	4.0 to 5.0	Greater than or equal to 60	4.0 to 5.0

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 2-7 through 2-10. 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. AASHTO's *A Policy on Geometric Design of Streets and Highways* has additional discussion on ramp gradients.

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.

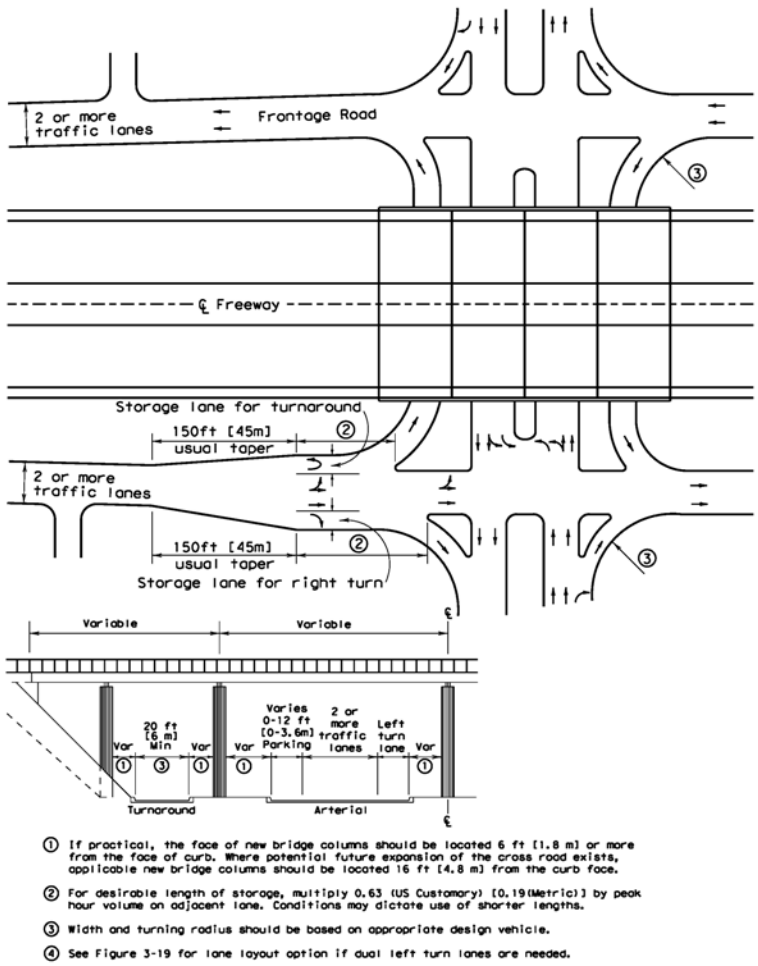
Where there is considerable demand for frequent ingress and egress, as in and near the business district of large cities, a collector-distributor road, continuous for some distance, should be provided.

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 3-38 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.



TYPICAL DIAMOND INTERCHANGE WITH FRONTAGE ROAD

Figure 3-38. Typical Diamond Interchange With Frontage Road. Click here to see a PDF of the image.

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.

Guidelines for the planning and designs of HOV facilities are given in AASHTO's *Guide for the Design of High Occupancy Vehicle Facilities* and in the HOV Systems Manual, National Cooperative Highway Research Program Report 414, Transportation Research Board, National Research Council, Washington, D.C., 1998.

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-Chavala and Ragland, *TCRP Report 17, Integration of Light Rail Transit into City Streets*, Transportation Research Board, National Academy Press, Washington, D.C., 1996.

Chapter 4 — Non-Freeway Rehabilitation (3R) Design Criteria

Contents:

Section 1 — Purpose

Overview

The basic purposes of resurfacing, restoration or rehabilitation (3R) construction 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
- 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
- urban frontage roads, Table 4-5.

Table 4-1: 3R Design Guidelines for Rural Multilane Highways (Nonfreeway)[a]

(US Customary)			
Design Element	**Highway Class**		
-	**6-Lane Divided**	**4-Lane Divided**	**4-Lane Undivided**
Design Speed[b]	50 mph	50 mph	50 mph
Lane Width	11 ft	11 ft	11 ft
Outside Shoulder	4 ft	4 ft	4 ft
Inside Shoulder	4 ft	2 ft	N/A
Turn Lane Width[c]	10 ft	10 ft	N/A
Horizontal Clearance	16 ft	16 ft	16 ft
Bridges[d]: Width to be retained	42 ft	28 ft	52 ft

[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.

NOTE: Online users can view the metric version of this table in PDF format.

Table 4-2: 3R Design Guidelines for Rural Two-Lane Highways[a]

(US Customary)			
Design Element	Current Average Daily Traffic		
-	0 – 400	400 -1500	1500 or more
Design Speed[b]	30 mph	30 mph	40 mph
Shoulder Width	0 ft	1 ft	3 ft
Lane Width	10 ft	11 ft	11 ft
Surfaced Roadway	20 ft	24 ft	28 ft
Turn Lane Width[c]	10 ft	10 ft	10 ft
Horizontal Clearance	7 ft	7 ft	16 ft
Bridges[d]: Width to be retained	20 ft	24 ft	24 ft[e]

[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 current ADT exceeding 2000, minimum width of bridge to be retained is 28 ft.

NOTE: Online users can view the metric version of this table in PDF format.

Table 4-3: 3R Design Guidelines for Urban Streets[a] All Functional Classes

(US Customary)	
Design Element	Guideline
Design Speed[b]	30 mph
Lane Width	10 ft
Turn Lane Width[c]	10 ft
Parallel Parking Lane Width	7 ft
Curb Offset	0 ft
Shoulders[d,e]	2 ft
Horizontal Clearance	To back of curb or outside edge of shoulder
Bridges: Width to be retained	Approach roadway, not including shoulders

[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] Minimally 1 ft of shoulder surfaced where lane width is 10 ft thereby providing a 22 ft surfacing width.
[e] Applicable to uncurbed streets.

NOTE: Online users can view the metric version of this table in PDF format.

Design Values

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 suggested by accident history, 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. Substantial changes in existing horizontal and/or vertical alignment are considered reconstruction. These projects should be developed to reconstruction standards.

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 accident locations and potential accident sites 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 accident 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.

- 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, or installing traffic control devices and pavement markings.
- 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 accident problems. These may be used at the designer's option to meet the particular needs of the project.

- Maintenance personnel are normally more familiar with a particular route and can point out problem areas to the designer based on their experiences. These individuals frequently "work" accident locations and are called upon to perform corrective work necessitated by accidents.
- A traffic accident analysis can be conducted from reports generated using the Traffic Accident Information and Hazard Elimination Program volume of the Traffic Operations Manual. Additional information is available from the Traffic Operations Division or from District Traffic personnel.
- Accident locations can be hand plotted on a straight-line drawing of the roadway in question. If a location or type-of-accident cluster is found, a more detailed review should be undertaken to determine potential causes of the accidents. If similar causes appear frequently, corrective measures should be designed into the 3R project.

A summary of the accident evaluation should accompany the submission. This evaluation should document the presence, or absence, of any major deficiencies which may contribute to accident 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 is the ADT and the character (truck %, recreational use, local traffic, etc.) of the traffic using the roadway?
- What is the accident 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*, 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 horizontal clearances 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 horizontal clearances. Consideration may also be necessary for trees, utility poles or other obstacles where these features are indicated significantly in an accident 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 will generally be installed to length 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 horizontal clearances 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 horizontal clearances given in this chapter are satisfied. Where the horizontal clearances 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 horizontal clearances 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[a]

(US Customary)		
Design Element	Current Average Daily Traffic	
-	0 - 1500	1500 or more
Design Speed	30 mph	40 mph
Lane Width	10 ft	11 ft
Shoulder Width Two-Way Operation	1 ft	3 ft
Inside Shoulder Width One-Way Operation	1 ft	2 ft
Outside Shoulder Width One-Way Operation	1 ft	4 ft
Horizontal Clearance	7 ft	16 ft
Bridges[b]: Width to be retained	22 ft	24 ftc

Table 4-4: 3R Design Guidelines for Rural Frontage Roads[a]

(US Customary)
[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] 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. [c] For current ADT exceeding 2000, minimum width of bridge to be retained is 28 ft.

NOTE: Online users can view the metric version of this table in PDF format.

Table 4-5: 3R Design Guidelines for Urban Frontage Roads[a]

(US Customary)	
Design Element	All Traffic Volumes
Design Speed	30 mph
Lane Width	10 ft
Shoulder Width	0 ft inside 2 ft outside
Horizontal Clearance	Back of curb or edge of shoulder
Curb Offset	1 ft either side
Bridges: Width to be retained	Approach roadway not including shoulders
[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.	

NOTE: Online users can view the metric version of this table in PDF format.

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 an accident 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.

Addtitional information on bridge rehabilitation may be obtained by referencing the Bridge Project Development Manual.

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

	Minimum	Desirable
Design Speed	See Table 4-2	
Horizontal Clearance	See Table 4-2	
Lane Width	11 ft	12 ft
Shoulder Width	3 ft[a]	8 - 10 ft
Passing Lane Length	1 mi	1.5 - 2 mi[b]

[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 = 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.

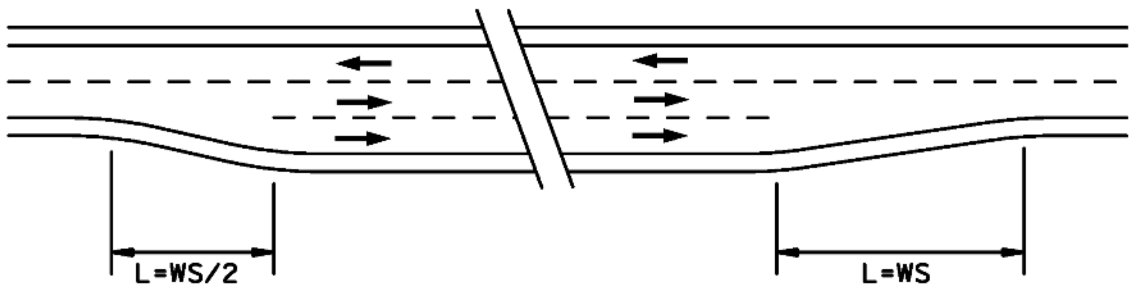


Figure 4-1. Opening and Closing a Passing Lane

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).

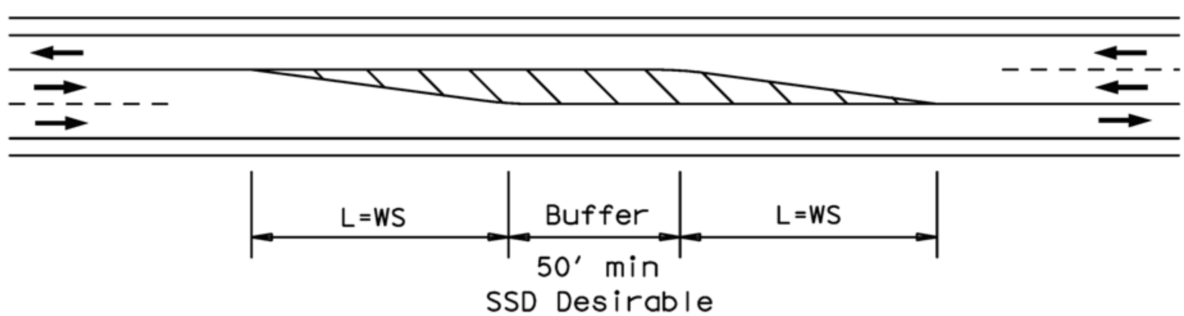


Figure 4-2. Closing the Passing Lane from One Direction to Another

When opening a passing lane in each direction (Figure 4-3), provide a taper length based on L= WS/2.

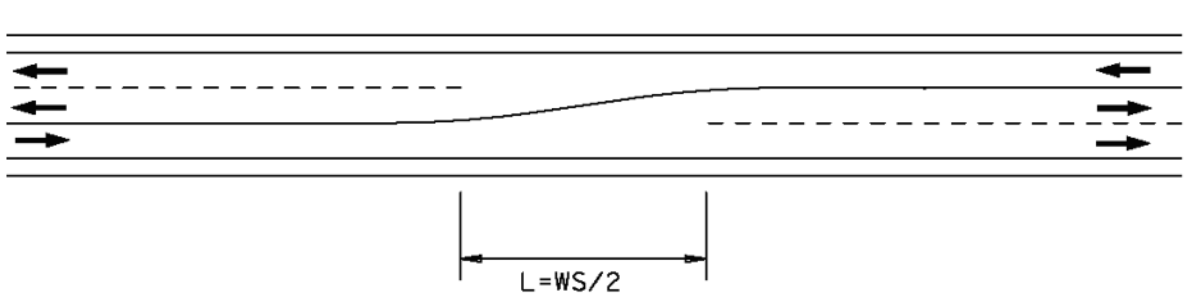


Figure 4-3. Opening the Passing Lane from One Direction to Another

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.

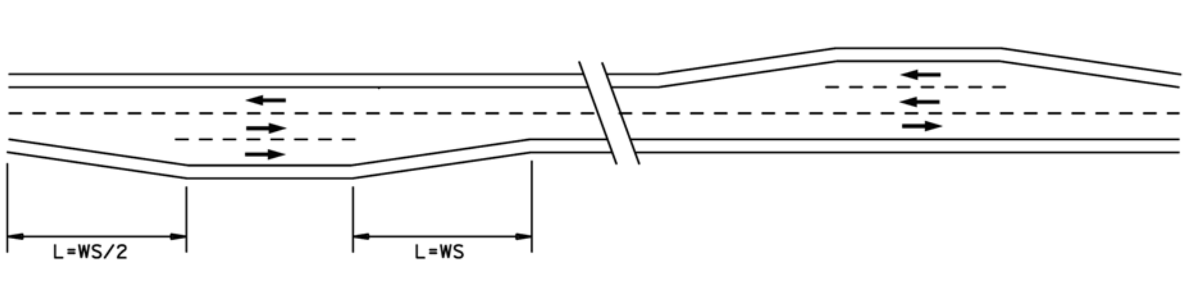


Figure 4-4. Separated Passing Lanes with Widening to the Outside of Roadway

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.

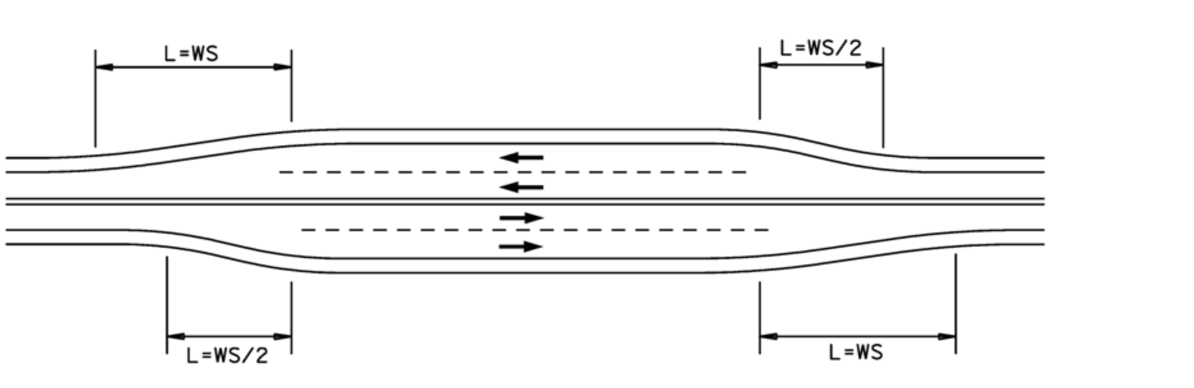


Figure 4-5. Side by Side Passing Lanes

Chapter 5 — Non-Freeway Resurfacing or Restoration Projects (2R)

Contents:

Section 1 — Overview

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

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
- 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*.

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 feet [15 m] adjacent to the bridge end, the roadway crown should match clear width across structure (24 feet [7.2 m]) plus additional width to accommodate approach guard fence.
- An appropriate transition (minimum length 50 feet [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 feet [6 m] surfacing width should be used for the 50 feet [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) 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 Texas Department of Transportation (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 are 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*.

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 two types of bicycle facilities described in the guide. These are bicycle lanes and bicycle 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. A bicycle 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.

Design Exceptions and Design Waivers for Bicycle Facilities

If the minimum requirements give in the *AASHTO Guide for the Development of Bicycle Facilities* for bicycle lanes cannot be met, these variances will be submitted as design exceptions to the Roadway Design Exception Committee. For new shared lanes on a signed, designated bicycle route, the minimum lane width shall be 14 ft [4.2 m]. Proposed widths less than 14 ft [4.2 m] will require approval by the Roadway Design Exception Committee.

If the minimum requirements given in the *AASHTO Guide for the Development of Bicycle Facilities* for bicycle paths cannot be met, these variances will be handled by design waivers at the district level.

Chapter 7 — Miscellaneous Design Elements

Contents:

Section 1 — Longitudinal Barriers

Overview

This section contains information regarding the following elements of longitudinal barriers:

- Concrete Barriers (Median and Roadside)
- Guardrail
- Attenuators (Crash Cushions).

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 2.5 ft [750 mm] and desirably 5 ft [1,500 mm] 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
- ease and cost of maintenance.

For more detailed information on the installation of various types of attenuators, refer to TxDOT's standard specifications and roadway standards.

Section 2 — Fencing

Right-of-way

The procedures for fencing highway right-of-way are in Right of Way Manual Vol. 2 - Right of Way Acquisition. 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 7-1: Use of Control of Access Fencing on Freeways

Condition	Type of Fence	Usual Location
Urban and suburban areas	Chain link fence of 4 ft [1.2 m] usual height or 6 ft [1.8 m] height where necessary for control of pedestrians	Variable[1]
Rural conditions where both large and small animals exist	Wire mesh fence with one or more strands of barbed wire	ROW line
Rural conditions where only large animals exist	Barbed wire fence with height of 4 ft or 5 ft [1.2 or 1.5 m]	ROW line
Control of Vehicles	Post and cable fence with closely spaced posts	Variable1
[1]Where frontage roads are provided, control of access fence, when used, should be placed in the outer separation approximately equidistant between the mainlanes and frontage roads and at least 30 ft [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 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, 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
- 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
- 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
- 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
  - kiss and ride
  - 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
- 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. Offstreet 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]
- 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 — Shoulder Texturing

Definition

Rumble strips are depressed or raised patterns used to provide auditory and tactile sensations to the driver to call attention to an upcoming change in conditions. Specifically, shoulder texturing is the use of rumble strips along the shoulder as a warning device to alert inattentive drivers that they are leaving the travelway.

Types of Shoulder Texturing

Milled-in. Milled-in rumble strips are effective types of shoulder texturing at reducing the number of single vehicle run-off-the-road accidents. Milled-in rumble strips are shallow depressions perpendicular to the edge line. Machinery specifically adapted for this type of work is required. A minimum shoulder width of 8 ft [2.4 m] is required for the outside shoulder to be milled. A minimum shoulder width of 4 ft [1.2 m] is required for the inside shoulder to be milled. Milled-in texturing produces sufficient stimuli to alert inattentive drivers, but does not affect the maneuverability capabilities of vehicles.

Rolled-in. Rolled-in strips may produce less noise and vibration than milled-in rumble stripes; however, rolled-in rumble strips are also effective at reducing the number of single vehicle run-off-the-road accidents. Rolled-in rumble strips are produced by half sections of pipe welded on a steel wheel roller at the appropriate spacing and rolled-in during the placement of hot mix asphaltic concrete pavement. Considerations in evaluating rolled-in texturing include 1) placement must be in coordination with other asphaltic concrete pavement construction, and 2) the temperature of the asphaltic concrete pavement is critical for achieving the proper depth without affecting the remaining surface. A minimum shoulder width of 8 ft [2.4 m] is required for the outside shoulder to be treated. A minimum shoulder width or 4 ft [1.2 m] is required for the inside shoulder to be treated.

Traffic Buttons. Traffic buttons placed along the edge line may also be used as shoulder texturing when milled-in or rolled-in texturing is not feasible. Buttons should be limited to roadways where there is insufficient pavement structure or shoulder width to accommodate either of the depressed texturing treatments and where the accident experience justifies the cost for placing and maintaining buttons. Buttons, however, may be used to supplement other shoulder texturing treatments when appropriate. Buttons may not be suitable where snow plows are used.

Raised Profile Thermoplastic Marking. Raised profile thermoplastic markings installed as the edge line may be used as shoulder texturing when rolled-in or milled-in texturing is not feasible. Raised profile thermoplastic markings used as the shoulder texturing treatment should be limited to roadways where there is insufficient pavement structure or shoulder width to accommodate either of the depressed texturing treatments. Raised profile thermoplastic markings, however, may be used to supplement other shoulder treatments, when appropriate.

Jiggle Bars. Jiggle bar tiles placed in a pattern perpendicular to the edge line may also be used as shoulder texturing when rolled-in or milled-in texturing is not feasible. The use of jiggle bars as shoulder texturing is not encouraged due to the level of auditory and tactile sensations caused by the jiggle bars and the high cost of installing the jiggle bars. Also, jiggle bars may not be suitable where snow plows are used. A minimum shoulder width of 8 ft [2.4 m] is required for the outside shoulder to be treated. A minimum shoulder width of 4 ft [1.2 m] is required for the inside shoulder to be treated.

Roadway Applications of Shoulder Texturing

For rural freeways and rural four-lane or more divided highways, the following guidelines are recommended:

- **Asphaltic Concrete Shoulders**: Rumble strips should be installed as part of new construction, reconstruction, and overlay projects on rural four-lane or more controlled and partially controlled access highways with asphaltic concrete shoulders.
- **Portland Cement Shoulders**: Rumble strips should be installed as part of new construction and reconstruction projects. If the concrete shoulder will be used in the near future as a permanent travel lane or a travel lane in a work zone, shoulder texturing should not be considered.

For rural four-lane or more undivided and rural two-lane highways, shoulder texturing on asphaltic concrete or portland cement shoulders is not recommended for these facilities except in special cases where a significant number of accidents, by frequency and percentage of total accidents, are run-off-the-road accidents and the installation of rumble strips is determined to be cost beneficial. The accident history, along with consideration of shoulder use by traffic, mail carriers, bicyclists, and/or farm equipment should be evaluated. If the concrete shoulder will be used in the near future as a permanent travel lane or a travel lane in a work zone, shoulder texturing should not be considered.

For urban highways, shoulder texturing on asphaltic concrete or portland cement shoulders is not recommended.

Bicyclists. When installing shoulder treatments, appropriate riding space for bicyclists needs to be a consideration. The standard details for shoulder texturing treatments provide appropriate riding space.

Placement. 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 (i.e., milled-in or rolled-in) shall not be placed across bridge decks.

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
- 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:

- the type and frequency of use by large vehicles
- consequences of encroachment into other lanes or the roadside
- availability of right-of-way
- 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.

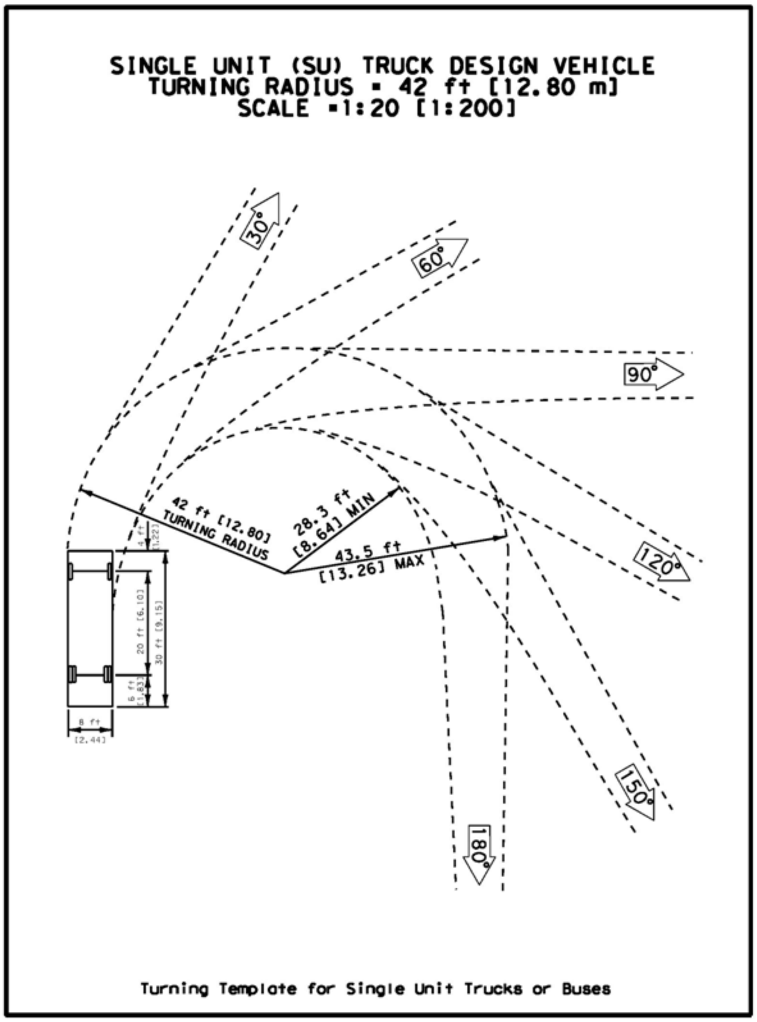


Figure 7-1. Turning Template for Single Unit Trucks or Buses, (not to scale). Click here to see a PDF of the image.

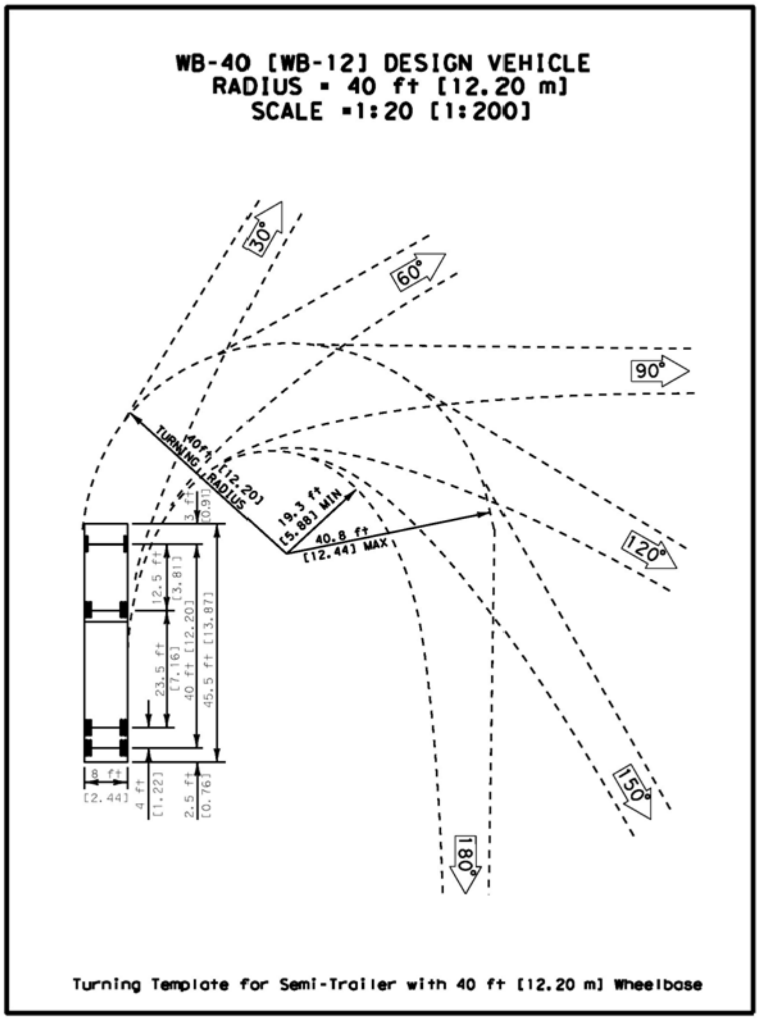


Figure 7-2. Turning Template for Semi-Trailer with 40 ft [12.20 m] Wheelbase, (not to scale). Click here to see a PDF of the image.

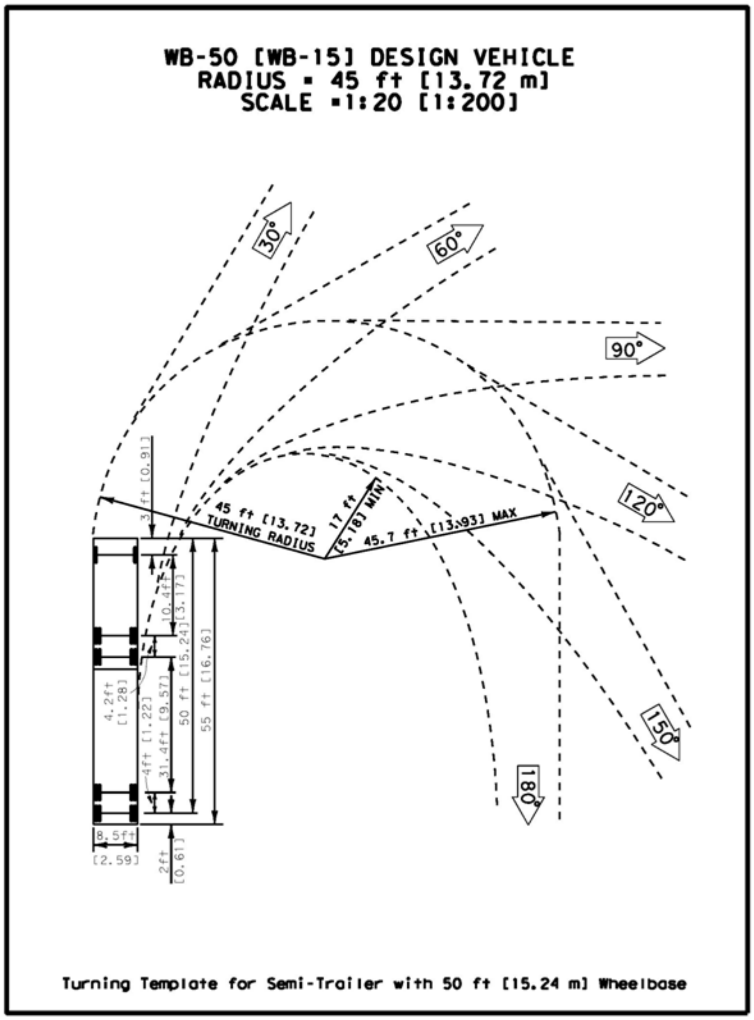


Figure 7-3. Turning Template for Semi-Trailer with 50 ft [15.24 m] Wheelbase, (not to scale). Click here to see a PDF of the image.

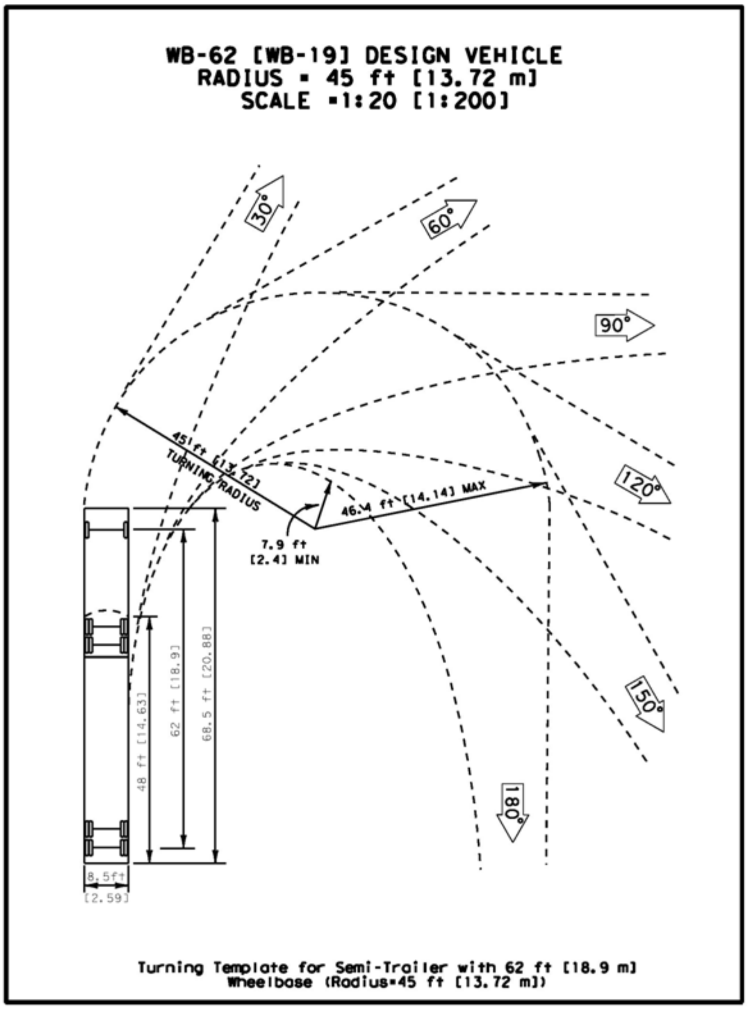


Figure 7-4. Turning Template for Semi-Trailer with 62 ft [18.9 m] Wheelbase, (not to scale). Click [here](#) to see a PDF of the image.

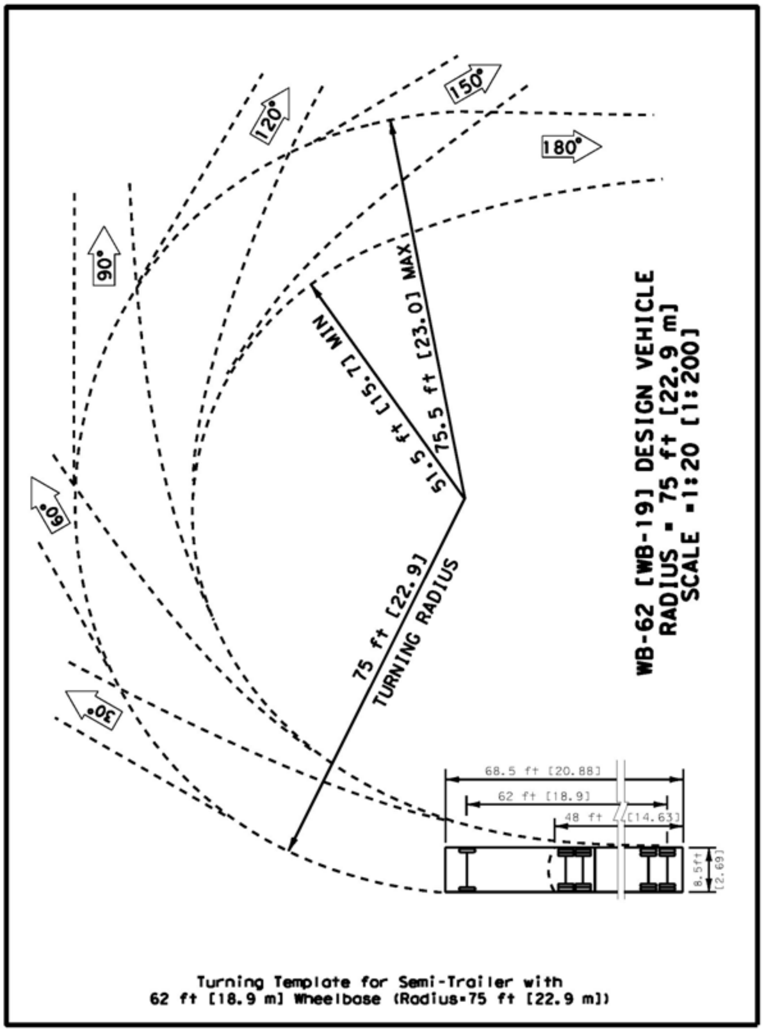


Figure 7-5. Turning Template for Semi-Trailer with 62 ft [18.9 m] Wheelbase (Radius = 75 ft [22.9 m], (not to scale). Click here to see a PDF of the image.

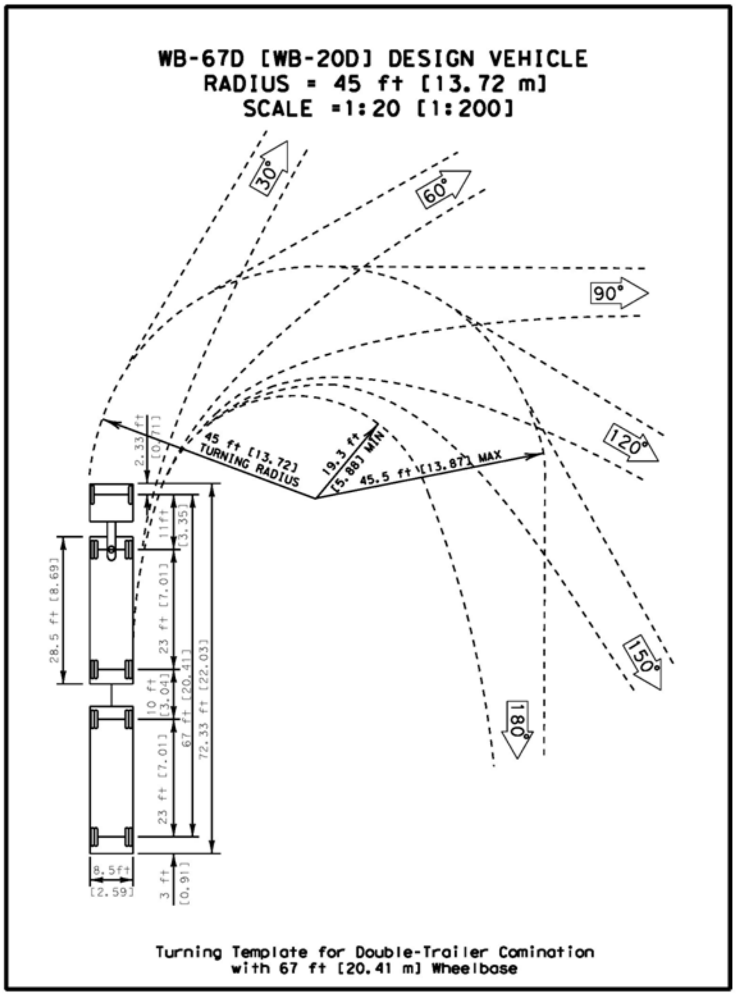


Figure 7-6. Turning Template for Double-Trailer Combination with 67 ft [20.41 m] Wheelbase, (figure not to scale). Click here to see a PDF of the image

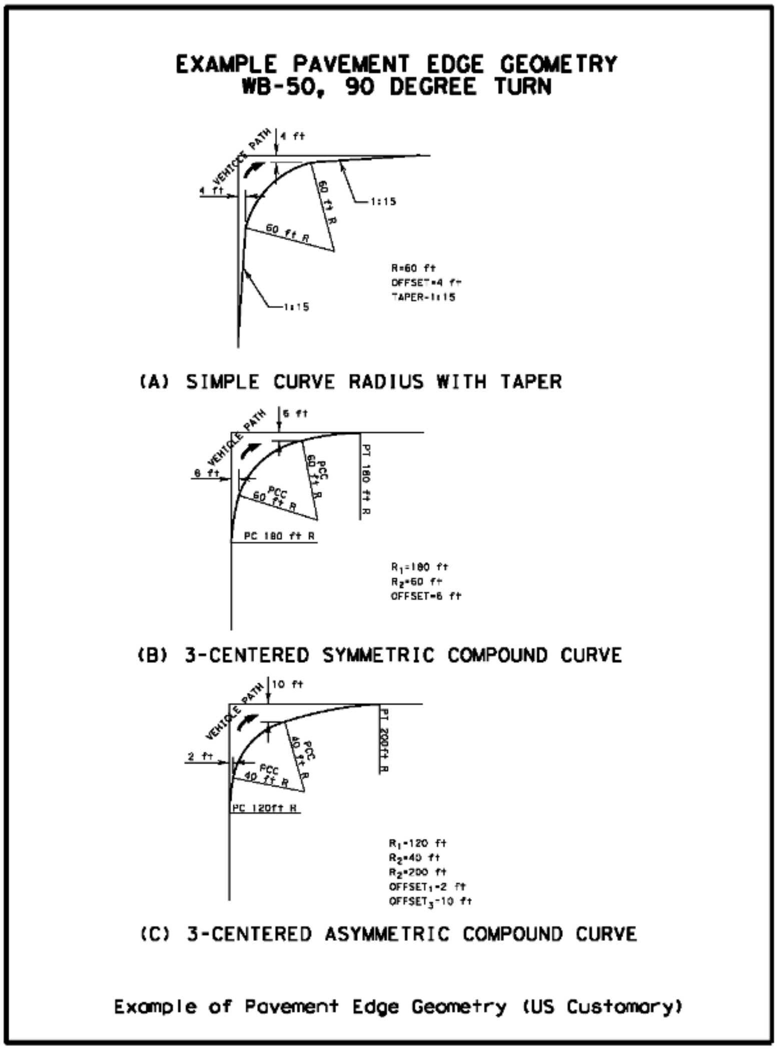


Figure 7-7. (US). Example of Pavement Edge Geometry (US Customary).

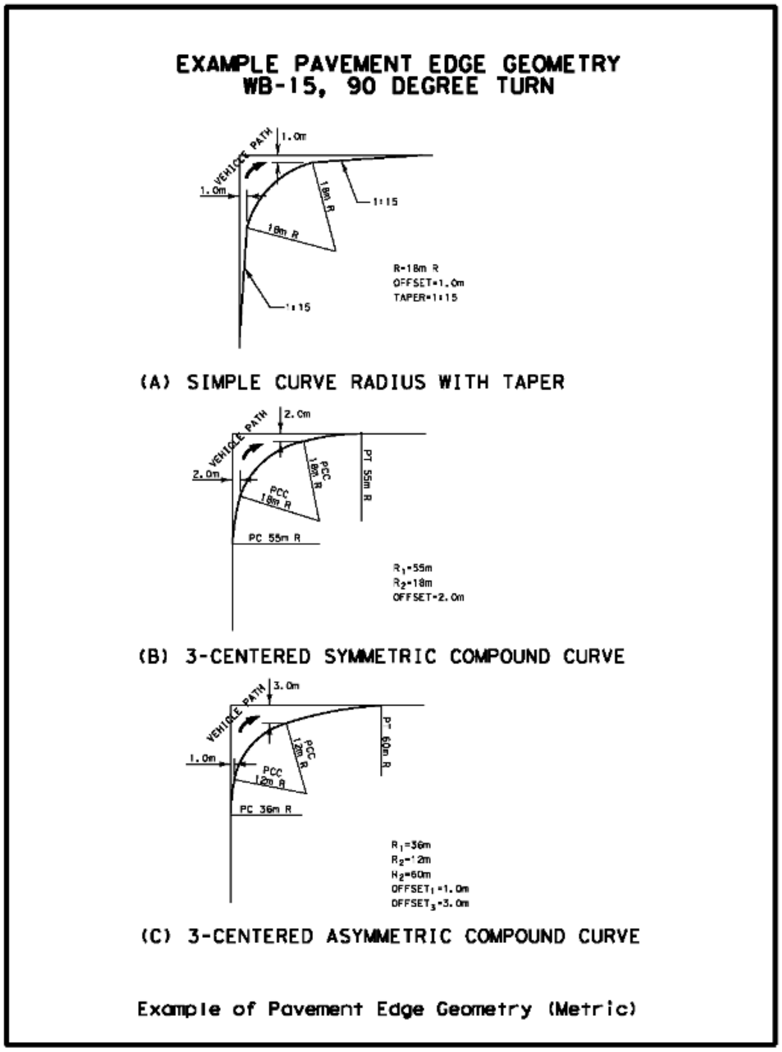


Figure 7-8. (M). Example of Pavement Edge Geometry (Metric).

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 ft^2 [4.5 m^2] in urban and 75 ft^2 [7.0 m^2] for rural conditions (100 ft^2 [9.0 m^2] 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.

(US Customary)

Angle of Turn[1] (degrees)	**Design Vehicle**	**Simple Curve Radius**	**Simple Curve Radius with Taper**			**3-Centered Compound Curve, Symmetric**		**3-Centered Compound Curve, Asymmetric**	
-	-	(ft)	Radius (ft)	Offset (ft)	Taper	Radii (ft)	Offset (ft)	Radii (ft)	Offset (ft)
60	P	40	-	-	-	-	-	-	-
-	SU	60	-	-	-	-	-	-	-
-	WB-40	90	-	-	-	-	-	-	-
-	WB-50	150	120	3.0	15:1	200-75-200	5.5	200-75-275	2.0-7.0
75	P	35	25	2.0	10:1	100-75-100	2.0	-	-
-	SU	55	45	2.0	10:1	120-45-120	2.0	-	-
-	WB-40	-	60	2.0	15:1	120-45-120	5.0	120-45-195	2.0-6.5
-	WB-50	-	65	3.0	15:1	150-50-150	6.5	150-50-225	2.0-10.0
90	P	30	20	2.5	10:1	100-20-100	2.5	-	-
-	SU	50	40	2.0	10:1	120-40-120	2.0	-	-
-	WB-40	-	45	4.0	10:1	120-40-120	5.0	120-40-200	2.0-6.5
-	WB-50	-	60	4.0	15:1	180-60-180	6.5	120-40-200	2.0-10.0
105	P	-	20	2.5	-	100-20-100	2.5	-	-
-	SU	-	35	3.0	-	100-35-100	3.0	-	-
-	WB-40	-	40	4.0	-	100-35-100	5.0	100-55-200	2.0-8.0
-	WB-50	-	55	4.0	15:1	180-45-180	8.0	150-40-210	2.0-10.0
120	P	-	20	2.0	-	100-20-100	2.0	-	-
-	SU	-	30	3.0	-	100-30-100	3.0	-	-
-	WB-40	-	35	5.0	-	120-30-120	6.0	100-30-180	2.0-9.0
-	WB-50	-	45	4.0	15:1	180-40-180	8.5	150-35-220	2.0-12.0

(US Customary)

Angle of Turn[1] (degrees)	Design Vehicle	Simple Curve Radius	Simple Curve Radius with Taper	3-Centered Compound Curve, Symmetric	3-Centered Compound Curve, Asymmetric
[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.					

(Metric)

Angle of Turn[1] (degrees)	Design Vehicle	Simple Curve Radius	Simple Curve Radius with Taper			3-Centered Compound Curve, Symmetric		3-Centered Compound Curve, Asymmetric	
-	-	(m)	Radius (m)	Offset (m)	Taper	Radii (m)	Offset (m)	Radii (m)	Offset (m)
60	P	12	-	-	-	-	-	-	-
-	SU	18	-	-	-	-	-	-	--
-	WB-12	28	-	-	-	-	-	-	-
-	WB-15	45	29	1.0	15:1	60-23-60	1.7	60-23-84	0.6-2.0
75	P	11	8	0.6	10:1	30-8-30	0.6	-	-
-	SU	17	14	0.6	10:1	36-14-36	0.6	-	-
-	WB-12	-	18	0.6	15:1	36-14-36	1.5	36-14-60	0.6-2.0
-	WB-15	-	20	1.0	15:1	45-15-45	2.0	45-15-69	0.6-3.0
90	P	9	6	0.8	10:1	30-6-30	0.8	-	-
-	SU	15	12	0.6	10:1	36-12-36	0.6	-	-
-	WB-12	-	14	1.2	10:1	36-12-36	1.5	36-12-60	0.6-2.0
-	WB-15	-	18	1.2	15:1	55-18-55	2.0	36-12-60	0.6-3.0
105	P	-	6	0.8	8:1	30-6-30	0.8	-	-
-	SU	-	11	1.0	10:1	30-11-30	1.0	-	-
-	WB-12	-	12	1.2	10:1	30-11-30	1.5	30-17-60	0.6-2.5
-	WB-15	-	17	1.2	15:1	55-14-55	2.5	45-12-64	0.6-3.0
120	P	-	6	0.6	10:1	30-6-30	0.6	-	-
-	SU	-	9	1.0	10:1	30-9-30	1.0	-	-
-	WB-12	-	11	1.5	8:1	36-9-36	2.0	30-9-55	0.6-2.7
-	WB-15	-	14	1.2	15:1	55-12-55	2.6	45-11-67	0.6-3.6

(Metric)

Angle of Turn[1] (degrees)	Design Vehicle	Simple Curve Radius	Simple Curve Radius with Taper	3-Centered Compound Curve, Symmetric	3-Centered Compound Curve, Asymmetric
[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.
- 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 three-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.

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

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 in 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.

- 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 and Design Variances, 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 these facilities are as described 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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-1: Stopping Sight Distance (US Customary)

Design Speed (mph)	Brake reaction distance (ft)	Braking distance on level (ft)	Stopping Sight Distance	
-	-	-	**Calculated (ft)**	**Design (ft)**

Table 8-1: Stopping Sight Distance (US Customary)

Design Speed (mph)	Brake reaction distance (ft)	Braking distance on level (ft)	Stopping Sight Distance	
85	312.4	693.5	1005.8	1010
90	330.8	777.5	1108.2	1110
95	349.1	866.2	1215.4	1220
100	367.5	959.8	1327.3	1330
(Metric)				
Design Speed (km/h)	**Braking distance on level (m)**	**Brake reaction distance (m)**	**Stopping Sight Distance**	
			Calculated (m)	**Design (m)**
140	97.3	224.8	322.1	325
150	104.3	258.1	362.3	365
160	111.2	293.6	404.8	405
NOTE: brake reaction distance predicated on a time of 2.5s; deceleration rate 11.2 ft/sec[3.4 m/sec]				

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 3 percent, regardless of initial speed. Grades above 2 percent may affect truck traffic depending on length of grade.

Table 8-2 summarizes the maximum grade controls in terms of design speed.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-2: Maximum Grades (US Customary)

Type of Terrain	Design Speed			
-	**85**	**90**	**95**	**100**
Level	2-3	2-3	2-3	2-3
Rolling	4	4	4	4
(Metric)				
-	140	150	160	--
Level	2-3	2-3	2-3	--
Rolling	4	4	4	--

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.

NOTE: Online users can click here to see the below tables in PDF format.

Table 8-3: Horizontal Curvature Highways and Connecting Roadways with Superelevation (US Customary [based on emax = 6%])

Design Speed (mph)	**Usual Min. Radius of Curve (ft)**	**Absolute Min.[1] Radius of Curve (ft)**
85	5615	3710
90	6820	4500
95	8285	5470
100	10100	6670
(Metric [based on emax = 6%])		
Design Speed (km/h)	Usual Min. Radius of Curve (m)	Absolute Min.[1] Radius of Curve (m)
140	1800	1190
150	2440	1615
160	3050	2020
1 Absolute minimum values should be used only where unusual design circumstances dictate.		

Table 8-3: Horizontal Curvature Highways and Connecting Roadways with Superelevation (US Customary [based on emax = 8%])

Design Speed (mph)	**Usual Min. Radius of Curve (ft)**	**Absolute Min.[1] Radius of Curve (ft)**
85	4865	3215
90	5845	3860
95	7010	4630
100	8420	5560
(Metric [based on emax = 8%])		
Design Speed (km/h)	Usual Min. Radius of Curve (m)	Absolute Min.[1] Radius of Curve (m)
140	1560	1030

Table 8-3: Horizontal Curvature Highways and Connecting Roadways with Superelevation (US Customary [based on emax = 8%])

Design Speed (mph)	Usual Min. Radius of Curve (ft)	Absolute Min.[1] Radius of Curve (ft)
150	2060	1365
160	2550	1680
[1]Absolute minimum values should be used only where unusual design circumstances dictate.		

NOTE: Online users can click here to see the below table in PDF format.

Table 8-4: Side Friction Factors and Running Speeds for Horizontal Curves

(US Customary)			(Metric)		
Design Speed (mph)	Side Friction Factor	Running Speed (mph)	Design Speed (km/h)	Side Friction Factor	Running Speed (km/h)
85	0.07	67	140	0.07	110
90	0.06	70	150	0.05	118[1]
95	0.05	75[1]	160	0.04	131[1]
100	0.04	82[1]			
[1]Values adjusted to eliminate negative friction on curve.					

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 85 shows the minimum curve radii without additional superelevation and an e_{max} of 8 percent.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-5: Horizontal Curvature of Highways without Superelevation[1] (US Customary)

Design Speed (mph)	Min. Radius (ft)
85	14700
90	16200
95	18800
100	22400
(Metric)	
Design Speed (km/h)	**Min. Radius (m)**

Table 8-5: Horizontal Curvature of Highways without Superelevation[1]
(US Customary)

Design Speed (mph)	Min. Radius (ft)
140	4680
150	5480
160	6750
[1]Normal crown (2%) maintained (e_{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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-6: Superelevation Rates for Horizontal Curves:
Superelevation Rate, e (6%), for Design Speed of
(US Customary)

Radius (ft)	85 mph	90 mph	95 mph	100 mph
23000	NC	NC	NC	NC
20000	NC	NC	NC	2.2
17000	NC	NC	2.2	2.6
14000	RC	2.3	2.6	3.2
12000	2.4	2.6	3.0	3.6
10000	2.8	3.1	3.6	4.3
8000	3.4	3.8	4.5	5.3
6000	4.5	5.0	5.8	R_{min} = 6670 ft
5000	5.2	5.8	R_{min} = 5470 ft	-
4000	5.9	R_{min} = 4500 ft	-	-
3500	R_{min} = 3710 ft	-	-	-
NC = Normal Crown		RC = Reverse Crown		e_{max} = 6%

NOTE: Online users can click here to see the below table in PDF format.

Table 8-6: Superelevation Rates for Horizontal Curves: Superelevation Rate, e (6%), for Design Speed of (Metric)

Radius (m)	140 km/h	150 km/h	160 km/h
7000	NC	NC	NC
5000	NC	2.1	2.6
3000	3.0	3.5	4.3
2500	3.5	4.2	5.1
2000	4.3	5.2	R_{min} = 2015 m
1500	5.5	R_{min} = 1610 m	-
1400	5.7	-	-
1300	5.9	-	-
1200	6.0	-	-
1000	R_{min} = 1190 m	-	-
NC = Normal Crown	RC = Reverse Crown		e_{max} = 6%

NOTE: Online users can click here to see the below table in PDF format.

Table 8-7: Superelevation Rates for Horizontal Curves: Superelevation Rate, e (8%), for Design Speed of (US Customary)

Radius (ft)	85 mph	90 mph	95 mph	100 mph
23000	NC	NC	NC	NC
20000	NC	NC	NC	2.2
17000	NC	NC	2.2	2.6
14000	2.1	2.3	2.7	3.2
12000	2.4	2.7	3.1	3.7
10000	2.9	3.2	3.7	4.5
8000	3.6	4.0	4.7	5.6
6000	4.8	5.3	6.2	7.4
5000	5.7	6.4	7.5	R_{min} = 5560 ft
4000	7.0	7.9	R_{min} = 4630 ft	-
3500	7.8	R_{min} = 3860 ft	-	-

Table 8-7: Superelevation Rates for Horizontal Curves:
Superelevation Rate, e (8%), for Design Speed of
(US Customary)

Radius (ft)	85 mph	90 mph	95 mph	100 mph
3000	R_{min} = 3215 ft	-	-	-
NC = Normal Crown		RC = Reverse Crown		e_{max} = 8%

NOTE: Online users can click here to see the below table in PDF format.

Table 8-7: Superelevation Rates for Horizontal Curves:
Superelevation Rate, e (8%), for Design Speed of

(Metric)			
Radius (m)	**140 km/h**	**150 km/h**	**160 km/h**
7000	NC	NC	NC
5000	NC	2.2	2.7
3000	3.1	3.6	4.5
2500	3.7	4.4	5.4
2000	4.6	5.5	6.7
1500	6.0	7.3	R_{min} = 1680 m
1400	6.4	7.8	-
1300	6.9	R_{min} = 1365 m	-
1200	7.4	-	-
1000	R_{min} = 1030 m	-	-
NC = Normal Crown		RC = Reverse Crown	- e_{max} = 8%

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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-8: Maximum Relative Gradient for Superelevation Transition

(US Customary)	(Metric)

Table 8-8: Maximum Relative Gradient for Superelevation Transition

Design Speed (mph)	Maximum Relative Gradient,%[1]	Equivalent Maximum Relative Slope	Design Speed (km/h)	Maximum Relative Gradient,%[1]	Equivalent Maximum Relative Slope
85	0.33	1:303	140	0.32	1:313
90	0.30	1:333	150	0.28	1:357
95	0.28	1:357	160	0.25	1:400
100	0.25	1:400	-	-	-
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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-9: Vertical Curves (US Customary)

Design Speed (mph)	Stopping Sight Distance (ft)	Crest Vertical Curves	Sag Vertical Curves
-	-	Design K	Design K
85	1010	473	260
90	1110	571	288
95	1220	690	319
100	1330	820	350
(Metric)			
Design Speed (km/h)	**Stopping Sight Distance (m)**	**Crest Vertical Curves**	**Sag Vertical Curves**
-	-	Design K	Design K
140	325	161	84
150	365	203	96
160	405	250	107

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

Horizontal Clearance

The horizontal clearance distances are shown in Table 8-10.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-10: Horizontal Clearance Distances (US Customary)

Design Speed (mph)	Horizontal Clearance Distance (ft)
85	80
90	80
95	90
100	100
(Metric)	
Design Speed (km/h)	**Horizontal Clearance Distance (m)**
140	24
150	28
160	30

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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-11 Earth Fill Slope Rates

Height of Fill	Usual Max[1] Slope Rate, Vertical:Horizontal	
-	Type of Terrain	
	Flat or Gently Rolling	Rolling
0 - 5 ft [0 - 1.5 m]	1V:8H	1V:6H
5 ft and over [1.5 m and over]	1V:6H	1V:6H

Table 8-11 Earth Fill Slope Rates

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. Subsequent discussions referring to ramps shall be construed to also include and apply to direct connections.

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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-12: Guide Values for Ramp/Connection Design Speed as Related to Highway Design Speed[1]

(US Customary)				
-	**Highway Design Speed (mph)**			
	85	90	95	100
Ramp Design Speed (mph):	-			
Upper Range (85%)	75	80	85	90
Mid Range (70%)	65	70	75	80
(Metric)				

Table 8-12: Guide Values for Ramp/Connection Design Speed as Related to Highway Design Speed[1]

(US Customary)			
-	Highway Design Speed (km/h)		
	140	150	160
Ramp Design Speed (km/h)	-		
Upper Range (85%)	120	130	140
Mid Range (70%)	110	120	130
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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-13: Ramp and Direct Connection Widths (US Customary)

-	Inside Shoulder Width (ft)	Outside Shoulder Width1 (ft)	Traffic Lanes(ft)
1-lane	8	10	14
2-lane	4	10	26
(Metric)			
-	**Inside Shoulder Width (m)**	**Outside Shoulder Width1 (m)**	**Traffic Lanes (m)**
1-lane	2.4	3.0	4.3
2-lane	1.2	3.0	7.9
1If 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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-14: Lengths of Exit Ramp Speed Change Lanes (US Customary)

Highway Design Speed (mph)	Minimum Length of Taper, T (ft)	Deceleration Length, D (ft) for Exit Curve Design Speed (mph)													
		Stop	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**
		Assumed Exit Curve Speed (mph)													
		0	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**	**48**	**52**	**55**	**58**	**61**
30	**Existing Criteria in Roadway Design Manual Figure 3-36**										--	--	--	--	--
35											--	--	--	--	--
40											--	--	--	--	--
45											--	--	--	--	--
50											--	--	--	--	--
55											--	--	--	--	--
60											185	--	--	--	--
65											225	185	--	--	--
70											270	225	190	--	--
75											310	265	235	195	--
80											335	310	275	240	200
85	345	650	630	615	595	575	550	510	475	440	355	310	275	240	200
90	360	695	675	660	645	625	600	555	525	490	400	355	325	285	250
95	370	780	760	745	725	705	680	640	605	570	450	405	370	335	295
100	425	900	880	865	850	830	805	760	730	695	530	485	455	415	375
											655	610	575	540	500
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.															

NOTE: Online users can click here to see the below table in PDF format.

Table 8-14: Lengths of Exit Ramp Speed Change Lanes (Metric)

Highway Design Speed (km/h	Minimum Length of Taper, T (m)	Deceleration Length, D (m) for Exit Curve Design Speed (km/h)											
		Stop	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**
		Assumed Exit Curve Speed (km/h)											
		0	**20**	**28**	**35**	**42**	**51**	**63**	**70**	**77**	**85**	**91**	**98**

Table 8-14: Lengths of Exit Ramp Speed Change Lanes (Metric)

50	**Existing Criteria in** Roadway Design Manual Figure 3-36									--	--	--	--
60										--	--	--	--
70										--	--	--	--
80										--	--	--	--
90										--	--	--	--
100										56	--	--	--
110										78	52	--	--
120										102	78	58	--
130										116	92	73	--
140	110	248	241	234	226	217	202	178	162	144	121	103	80
150	115	271	264	258	250	241	227	204	189	172	150	132	110
160	130	309	303	297	290	282	268	248	233	216	196	180	159

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.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-15: Lengths of Entrance Ramp Speed Change Lanes (US Customary)

Highway Design Speed (mph)	**Minimum Length of Taper, T (ft)**	**Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)**													
		Stop	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**
		Initial Speed (mph)													
		0	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**	**48**	**52**	**55**	**58**	
30	**Existing Criteria in Roadway Design Manual Figure 3-36**										--	--	--	--	--
35											--	--	--	--	--
40											--	--	--	--	--
45											--	--	--	--	--
50											--	--	--	--	--
55											--	--	--	--	--
60											--	--	--	--	--
65											132	--	--	--	--
70											331	70	--	--	--
75											545	287	74	--	--
80											771	516	306	79	--
85	345	2186	2154	2045	2006	1945	1828	1601	1426	1227	1009	757	550	326	84
90	360	2403	2379	2266	2233	2179	2065	1840	1668	1472	1259	1010	805	584	345
95	370	2786	2777	2658	2636	2593	2484	2264	2097	1906	1701	1459	1258	1042	808
100	425	3372	3385	3256	3250	3225	3123	2910	2751	2568	2375	2142	1949	1740	1514

Table 8-15: Lengths of Entrance Ramp Speed Change Lanes (US Customary)

NOTE: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.

NOTE: Online users can click here to see the below table in PDF format.

Table 8-15: Lengths of Entrance Ramp Speed Change Lanes (Metric)

Highway Design Speed (km/h	Minimum Length of Taper, T (m)	Acceleration Length, A (m) for Entrance Curve Design Speed (km/h)											
		Stop	20	30	40	50	60	70	80	90	100	110	120
		Initial Speed (km/h)											
		0	20	30	40	47	55	63	70	77	85	91	98
50	Existing Criteria in Roadway Design Manual Figure 3-36									--	--	--	--
60										--	--	--	--
70										--	--	--	--
80										--	--	--	--
90										--	--	--	--
100										--	--	--	--
110										48	--	--	--
120										156	46	--	--
130										218	109	--	--
140	110	703	687	793	652	624	572	507	438	350	245	155	37
150	115	819	806	945	776	750	700	646	581	492	392	305	190
160	130	977	987		940	928	877	787	726	657	570	496	397
Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 400 m.													

NOTE: Online users can click here to see the below table in PDF format.

Table 8-16: Speed Change Lane Adjustment Factors as a Function of a Grade (US Customary)

Design Speed of Roadway (mph)	Ratio of Length on Grade to Length on Level[1]							
	20	25	30	35	40	45	50	All Speeds
	3 to 4% Upgrade							3 to 4% Downgrade
85	1.62	1.69	1.75	1.80	1.89	1.99	2.10	0.56
90	1.66	1.73	1.80	1.86	1.96	2.08	2.20	0.55
95	1.71	1.78	1.85	1.92	2.03	2.17	2.30	0.54
100	1.75	1.83	1.90	1.98	2.10	2.26	2.40	0.52
	5 to 6 % Upgrade							5 to 6 % Downgrade

Table 8-16: Speed Change Lane Adjustment Factors as a Function of a Grade (US Customary)

85	2.39	2.51	2.64	2.94	3.15	3.73	4.28	0.46
90	2.50	2.64	2.77	3.10	3.33	4.00	4.65	0.45
95	2.62	2.76	2.91	3.27	3.51	4.26	5.03	0.44
100	2.74	2.89	3.04	3.43	3.69	4.53	5.40	0.42

(Metric)

Design Speed of Roadway (km/h)	**Ratio of Length on Grade to Length on Level**[1]					
	40	**50**	**60**	**70**	**80**	**All Speeds**
	3 to 4% Upgrade					**3 to 4% Downgrade**
140	1.57	1.67	1.81	1.79	1.90	0.55
150	1.60	1.70	1.86	1.83	1.94	0.53
160	1.63	1.73	1.90	1.86	1.99	0.52
	5 to 6 % Upgrade					**5 to 6 % Downgrade**
140	2.55	2.82	3.53	3.92	4.42	0.44
150	2.70	3.00	3.81	4.29	4.88	0.42
160	2.86	3.18	4.09	4.65	5.34	0.41

[1] Ratio in this table multiplied by length of acceleration distances gives length of acceleration distance on grade.

Distance between Successive Ramps

The minimum acceptable distance between ramps is dependent upon the merge, diverge, and weaving operations that take place between ramps and 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.

Appendix A — Metal Beam Guardrails

Contents:

Section 1 — Overview

Introduction

The objectives of this appendix are to make available data and guidelines for the use of roadside traffic barriers (typically metal beam guard fence) 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 Guardrail

Section 5 - End Treatment of Guardrail

Section 6 - Determining Length of Need of Barrier

Section 7 - Example Problems.

Section 2 — Barrier Need

Overview

Traffic barriers are needed only when the obstacle without the barrier is greater than the obstacle of the barrier itself.

Should a roadside obstacle exist, treatment should be considered in the following priority:

1. Eliminate the obstacle.
2. Redesign the obstacle so it can be safely traversed.
3. Relocate the obstacle outside the obstruction free zone to reduce the likelihood that it will be struck.
4. Treat the obstacle to reduce accident severity, i.e., use flush or yielding designs.
5. Shield the obstacle with a barrier (median barrier, roadside barrier, or crash cushion).
6. Delineate the obstacle if the above alternatives are not appropriate.

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
- 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 guardrail.

Table A-1: General Applications of Conditions for Roadside Barriers

Roadside Feature	Applications
Terrain Features:	
Steep Embankment Slope	hc[a], See Figure A-1
Rough Rock Cut	hc
Boulders	hc, dia. Exceeds 6 in [150 mm]
Water Body	hc, width exceeds 2 ft [600 mm], permanent
Lateral Drop-off	hc & steeper than 1V:1H and depth exceeds 2 ft [600 mm]
Side Ditches	hc & unsafe cross section[b]

Table A-1: General Applications of Conditions for Roadside Barriers

Roadside Feature	Applications
-	
Bridges:	
Parapet Wall/Wingwall/Bridge Rail End	hc & approaching traffic
Area Alongside Bridges	hc & approaching traffic
-	
Roadside Obstacles:	
Trees	hc & dia. Exceeds 6 in [150 mm]
Culvert Headwall	hc & size of opening exceeds 3 ft [900 mm] (w.o. safety grates only)
Wood Poles, Posts	hc & cross section/area exceeds 50 in^2 [32000 mm^2]
Bridge Piers, Abutments at Underpasses	hc
Retaining Walls	hc & not parallel to travelway

[a] hc - Within horizontal clearance for highway class and traffic volume conditions.
[b] For preferred ditch cross sections, see Side Ditches in Chapter 2

Where the prescribed length of the guardrail cannot be installed at a bridge end due to an intervening access point such as an intersecting roadway or driveway, the length of guardrail 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 wrapping the guardrail around the radius of the access location, terminating the guardrail prior to the access location with an appropriate end treatment and continuing the guardrail 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 guardrail length to accommodate access points will not require a design exception or a design waiver.

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 guardrail, an area free of obstructions should be provided by the designer beyond the toe of slope.

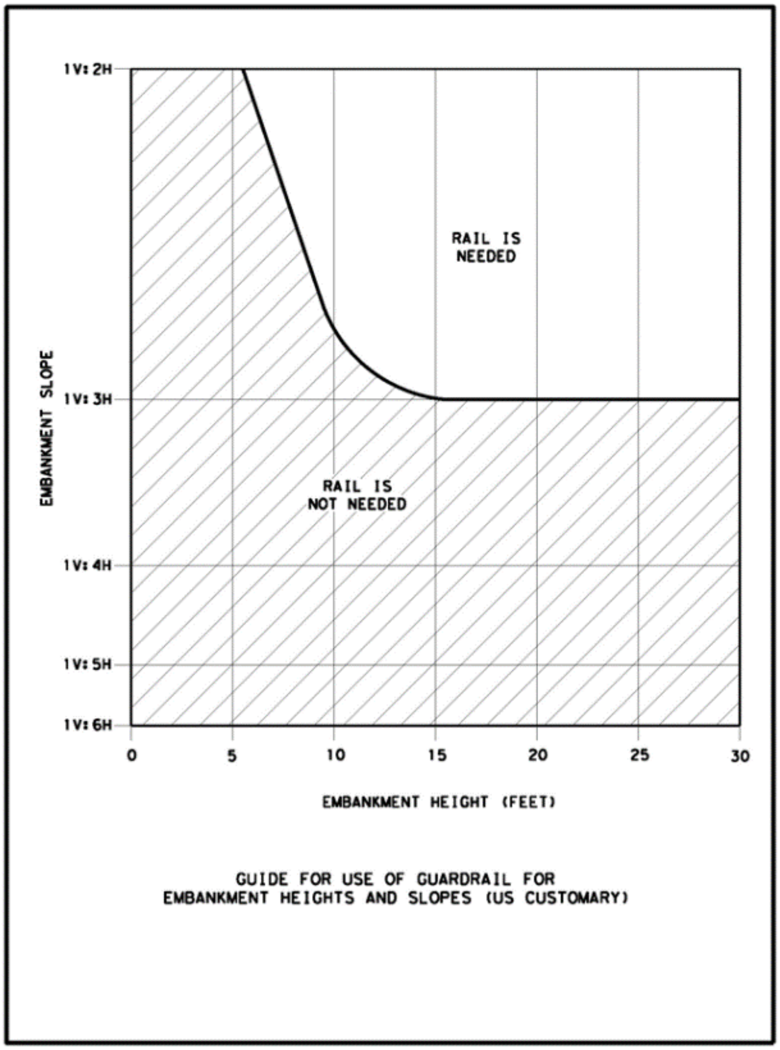


Figure A-1. (US). Guide for Use of Guardrail for Embankment Heights and Slopes (US Customary)

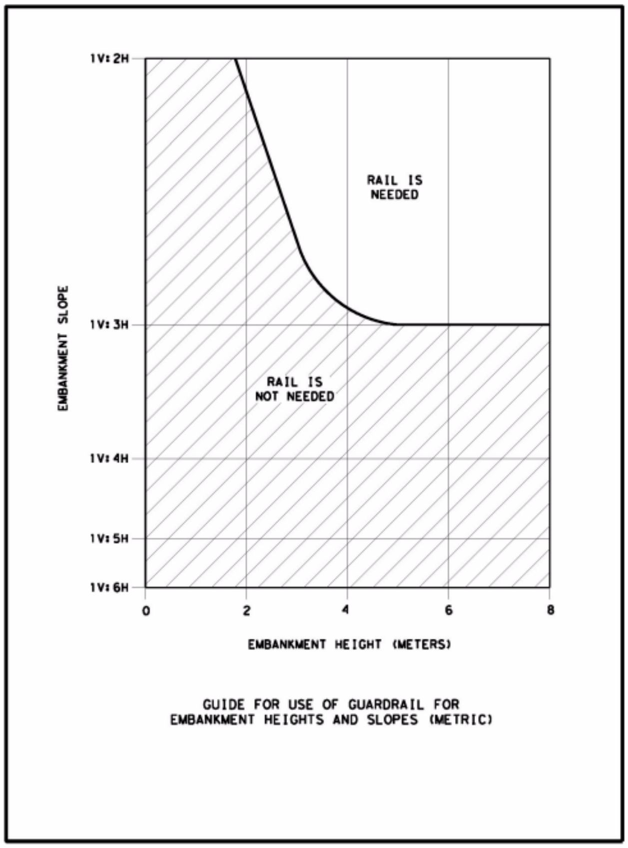


Figure A-2. (M). Guide for Use of Guardrail for Embankment Heights and Slopes (Metric)

Section 3 — Structural Considerations

Overview

Post spacing, rail shape and thickness, splice strength, post embedment, and rail anchorage are all important factors that influence the structural integrity of guardrail.

Post Spacing, Embedment, and Lateral Support

Typical post spacing is 6 ft- 3in [1905 mm] for guardrail. Where guardrail 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-2, 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.

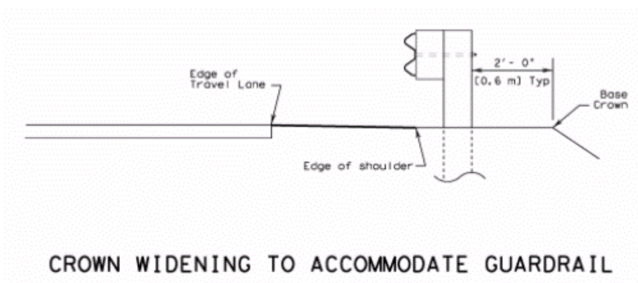


Figure A-3. Crown Widening to Accommodate Guardrail

Rail Element

Guardrail 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 21 in [550 mm] high as measured from shoulder surface, gutter pan, or widened crown to the center of the rail at the bolt.

Pavement overlays effectively reduce existing rail height. When rail height varies more than 3 in [75 mm] from standard height, steps should be taken to restore the rail to the standard dimension to reduce the possibility of vehicular vaulting or post snagging.

When raising existing rail, existing timber posts should be removed and replaced if rotted or otherwise deteriorated. Where existing timber posts are in good condition and embedded in cohesionless soils, the posts may be pulled (jacked) up to 4 in [100 mm]. These partially extracted posts should be restrained at ground line to preclude settlement, and the resultant rail height is increased up to 4 in [100 mm]. For cohesive soil conditions or where more than 4 in [100 mm] height increase is desired, timber posts should be removed and replaced.

Deflection Considerations

Guardrail is a flexible barrier system. The amount of dynamic deflection varies primarily with weight of impacting vehicle, its speed, and its encroachment angle. Guardrail 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 2.5 ft [750 mm] or more as diagrammed in Figure A-3. Where conditions permit, a barrier-to-obstacle distance of 5 ft [1500 mm] or more is desirable.

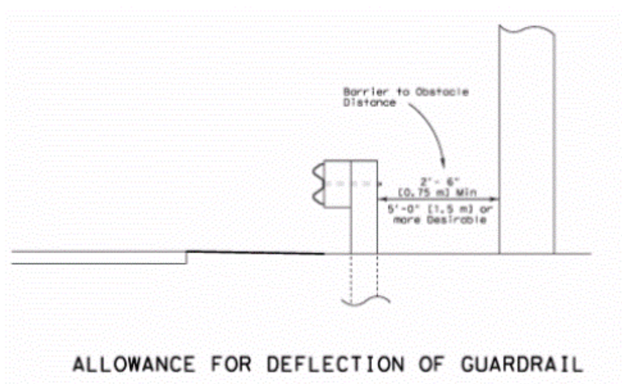


Figure A-4. Allowance for Deflection of Guardrail

Section 4 — Placement of Guardrail

Overview

The placement of guardrail pertains to the lateral and longitudinal position.

Lateral Placement at Shoulder Edge or Curb Face

Typically the face of rail is placed at the shoulder edge or curb face throughout most of its length as shown in Figure A-4.

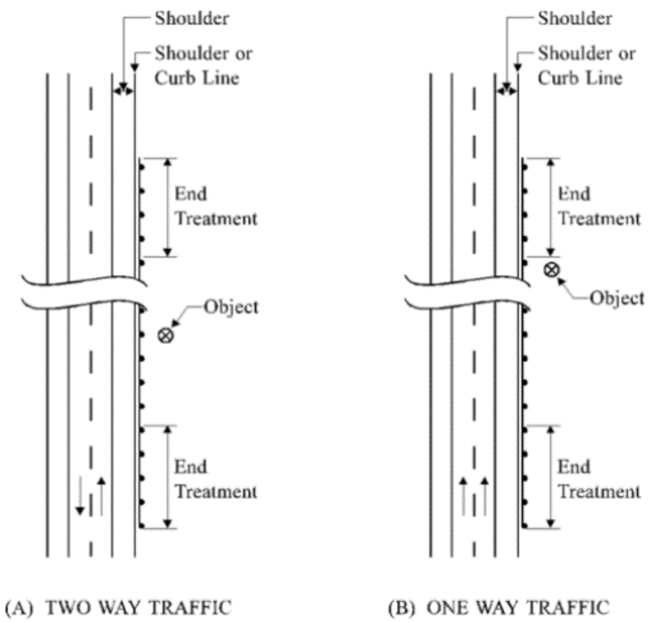


Figure A-5. Placement at Shoulder Edge or Curb Face

Guardrail 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 approximately 21 in [550 mm] above the gutter pan or roadway surface.

Lateral Placement Away From the Shoulder Edge

In certain instances it is desirable to place guardrail 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—point type obstacles such as 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 guardrail with respect to slope conditions. Guardrail 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.

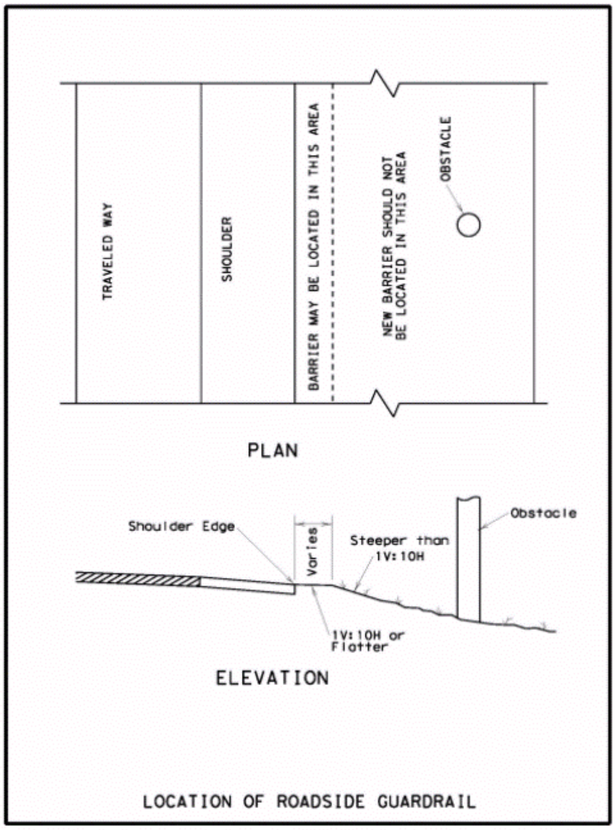


Figure A-6. Location of Roadside Guardrail. Click here to see a PDF of the image.

Section 5 — End Treatment of Guardrail

Overview

Guardrail 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 guardrail system. These approved end treatments shall be used unless the guardrail terminal is located on the downstream end [with respect to adjacent traffic—see Figure A-4 of the guardrail and outside the horizontal clearance for opposing traffic. In that case a twisted, buried end terminal anchor section without offset remains 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 horizontal clearance width are the primary variables influencing length of barrier need.

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-6, will be referred to an "area of concern."

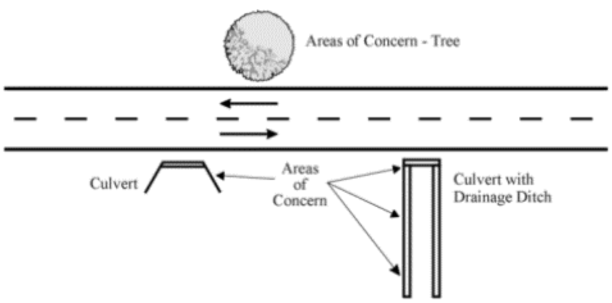


Figure A-7. Areas of Concern

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,
- the length of downstream barrier, L_d

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 horizontal clearance of opposing (northbound in Figure A-7) traffic as measured from the centerline pavement markings.

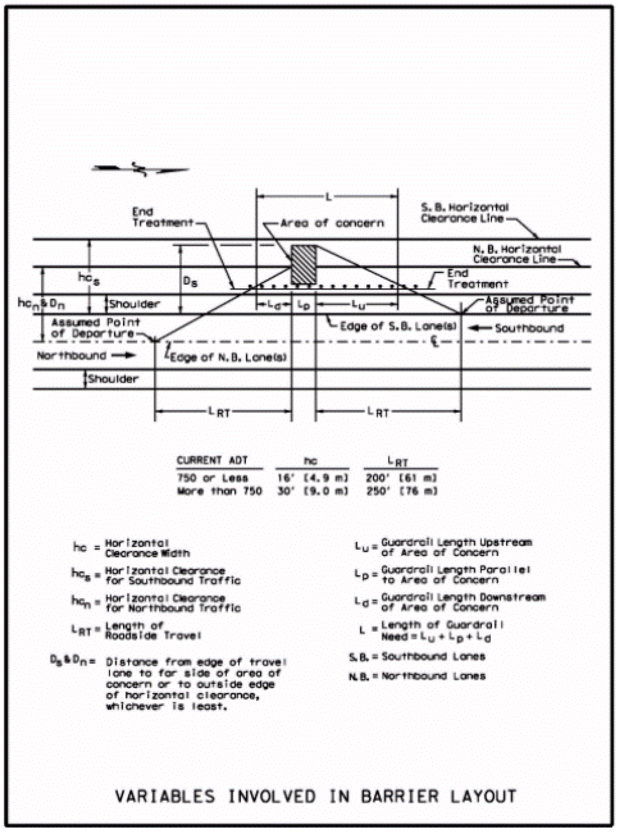


Figure A-8. Variables Involved in Barrier Layout. Click here to see a PDF of the image.

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 warrant minimum guardrail treatment where located slightly outside the horizontal clearance if geometric conditions (i.e., steep fill slope, outside of horizontal curvature, etc.) increase the likelihood of roadside encroachments.

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 horizontal clearance 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 horizontal clearance required is less than 16 ft [4.9 m] and the present ADT is 750 or less, use Equation A-1 for calculating the guardrail length of need.

US Customary:

$$L = 200 - \frac{200}{D} \times G$$

Where:

- *L* = Length of guardrail needed, ft
- *D* = Distance from edge of travel lane to far side of area of concern or to outside edge of horizontal clearance, whichever is least, ft
- *G* = Guardrail offset from edge of travel lane, ft

Metric:

$$[L = 61 - \frac{61}{D} \times G\ ]$$

Where:

- *L* = Length of guardrail needed, m
- *D* = Distance from edge of travel lane to far side of area of concern or to outside edge of horizontal clearance, whichever is least, m
- *G* = Guardrail offset from edge of travel lane, m

Equation A-1

For low volume conditions, if the horizontal clearance width (16 ft [4.9 m]) is met or exceeded, L=0.

For higher volumes, a horizontal clearance 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 horizontal clearance is greater than 16 ft [4.9 m]):

(US Customary):

$$L = 250 - \frac{250}{D} \times G$$

(Metric):

$$[L = 76 - \frac{76}{D} \times G]$$

Equation A-2

For high volume conditions, if the horizontal clearance width (30 ft [9 m]) is met or exceeded, L=0.

Using Design Equations to Determine Length of Guardrail

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.)
- 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.

The Example Problems section provides example problems and solutions using the design equations. The guardrail lengths produced by the equations should be rounded up to an even length of guardrail.

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 horizontal clearance 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.

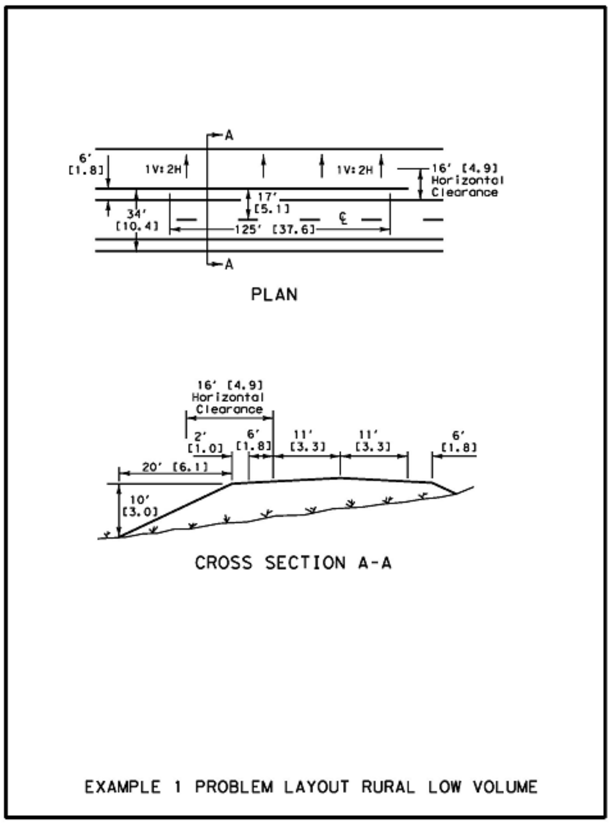


Figure A-9. Example 1 Problem Layout Rural Low Volume. Click here to see a PDF of the image.

Solution: From the information above and referring to Figure A-1 it is determined that a “rail is needed.” As shown in Figure A-7, the length of need is $L = L_u+L_p+L_d$. From the given information, L_p = 125 ft [38 m]. Because the ADT is less than 750, Equation A-1 is used to solve for L_u and L_d (if necessary).

For the upstream (westbound traffic) direction, the area of concern is the full (16 ft [4.9 m]) horizontal clearance width and the guardrail offset is 6 ft [1.8 m]. Substituting in Equation A-1.

(US Customary):

$$L_u = 200 - \frac{200}{16} \times 6 = 125\text{ft}$$

(Metric):

$$[L_u = 61 - \frac{61}{4.9} \times 1.8 = 38.6\text{m (use 38 m)}]$$

Equation A-3

As shown in Equation A-3, a placement of guardrail alongside the 6 ft [1.8 m]-wide shoulder results in L_u = 125 ft [38 m].

Referring to figure A-8, the length of guaradrail needed in the downstream (eastbound traffic) 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 horizontal clearance (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-9 including 125 ft [38 m] of guardrail adjacent to the obstacle plus 125 ft [38 m] shielding westbound traffic upstream of the obstacle. These lengths of need do not include end treatments.

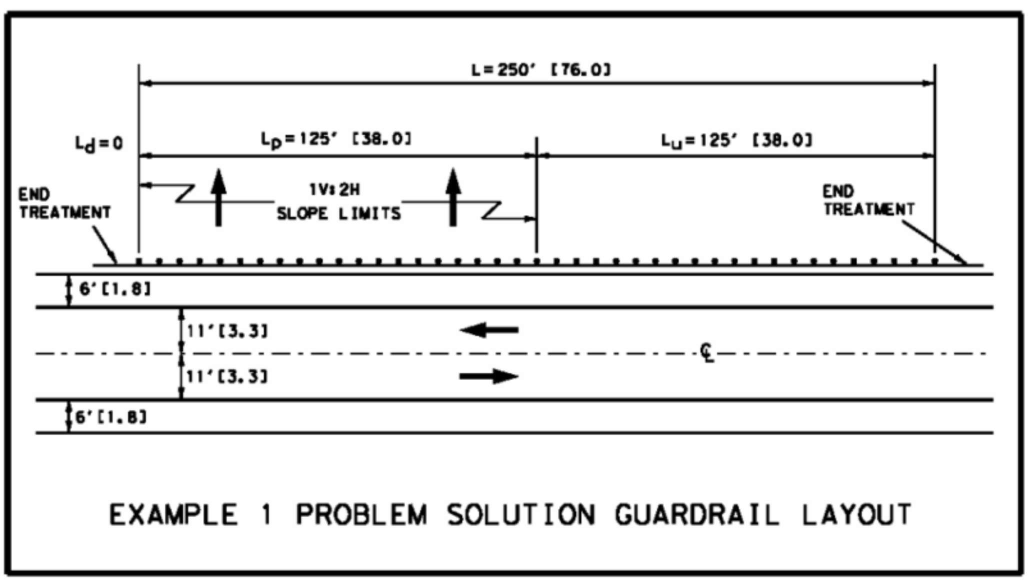


Figure A-10. Example 1 Problem Solution Guardrail Layout

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-10. The areas of concern are bridge bents located 5 ft [1.5 m] from the edge of shoulder. The side slopes are 1V:6H.

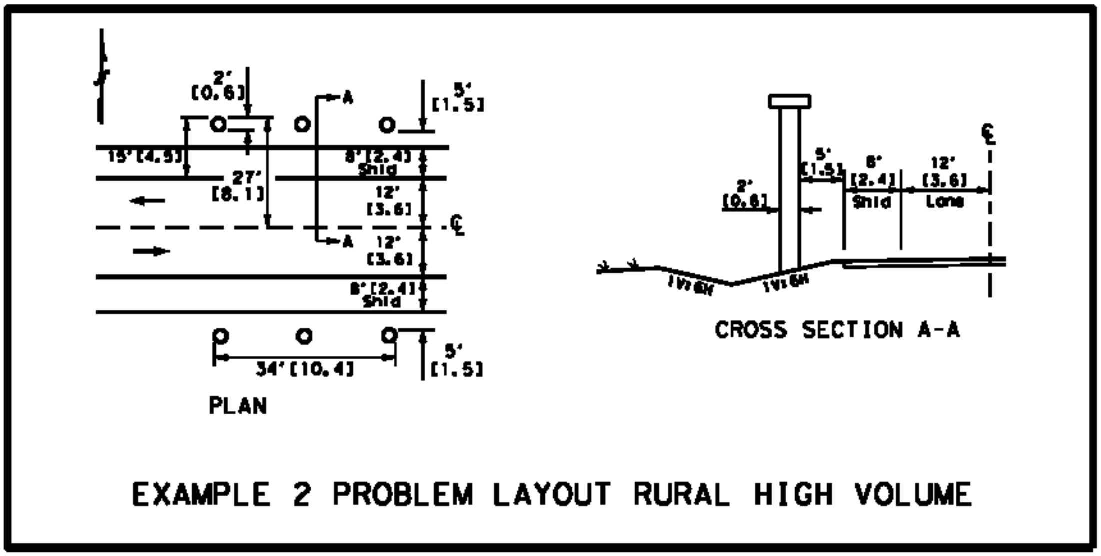


Figure A-11. Example 2 Problem Layout Rural High Volume.

Solution: Referring to Table A-1: General Applications of Conditions for Roadside Barriers bridge piers within the horizontal clearance (30 ft [9 m] in this case) warrant Guardrail placement for the northside of the roadway displayed in Figure A-10. As shown in Figure A-7 the length of need is $L = L_u + L_p + L_d$. Therefore, L_p is 34 ft [10.4 m] from the given (see Figure A-10) information. Because the ADT is greater than 750, Equation A-2 is used to find L_u and L_d (if necessary):

(US Customary):

$$L_u = 250 - \frac{250}{15} \times 8 = 116.5\text{ft}$$

(Metric):

$$[L_u = 76 - \frac{76}{4.5} \times 2.4 = 35.5\text{m}]$$

Equation A-4.

Substituting in the equation, the upstream length (L_u) is 116.5 ft [35.5 m] as shown in Equation A-4 if placement is at the shoulder edge.

The downstream (westbound traffic) length of guardrail is also determined by substituting into Equation A-2:

(US Customary):

$$L_d = 250 - \frac{250}{27} \times 20 = 65\text{ft}$$

Metric):

$$[L_d = 76 - \frac{76}{8.1} \times 6.0 = 19.7\text{m}]$$

Equation A-5

Substituting in the equation as shown in equation A-5, L_d is 65 ft [19.7 m] as shown in Equation A-5, based on the shoulder edge placement. For westbound traffic, the centerline is the edge of the travel lane and thus guardrail is placed 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 guardrail, 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 [62.3 m]; or, rounded to an even length of guardrail, 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-11.

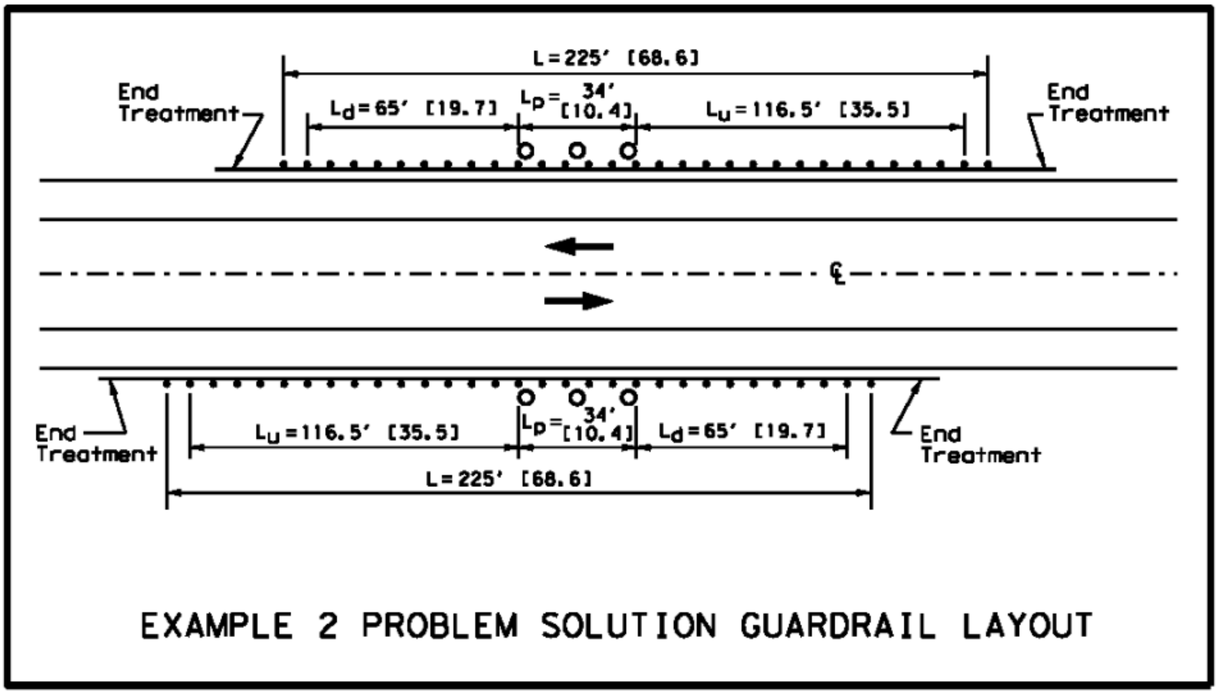


Figure A-12. Example 2 Problem Solution Guardrail Layout. Click here to see a PDF of the image.

Example Problem 3

Given: A divided (76 ft [22.8 m] median) highway with 4 ft [1.2 m] left and 10 ft [3.0 m] right shoulder widths is illustrated in Figure A-12. 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.

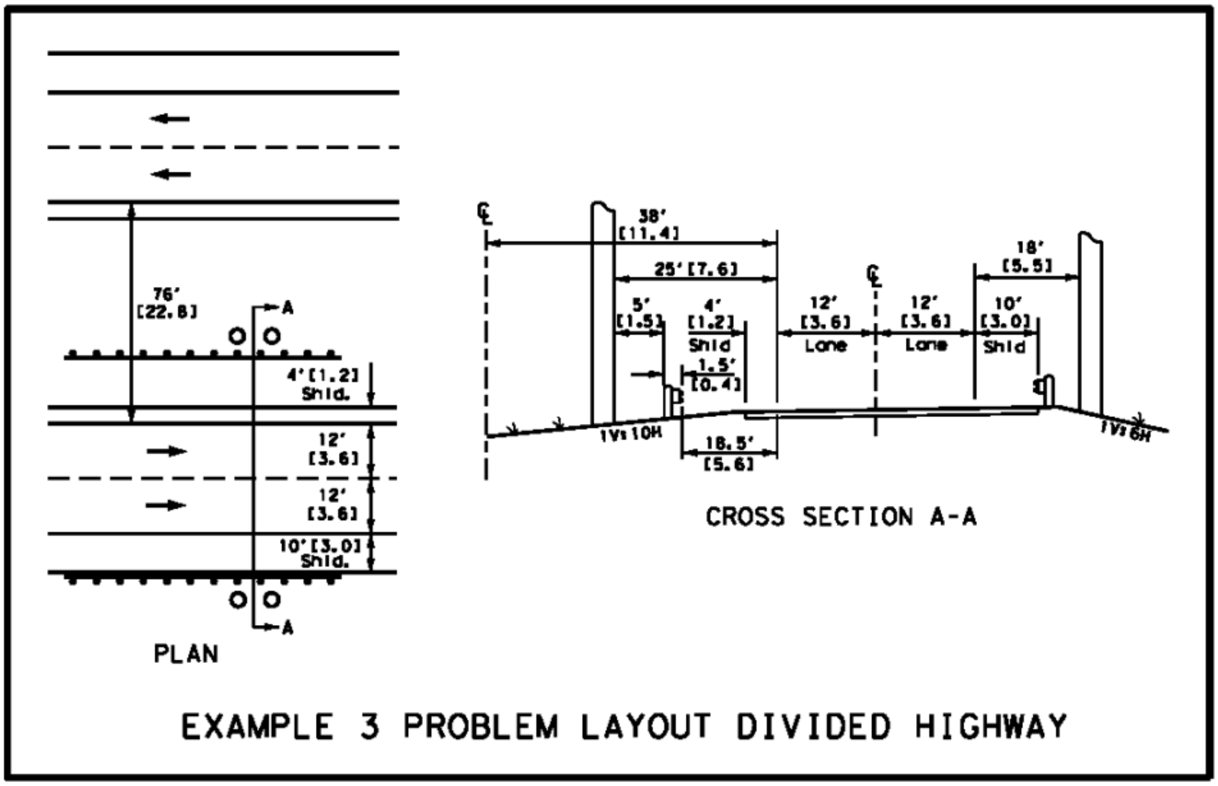


Figure A-13. Example 3 Problem Layout Divided Highway.

Solution: Crash cushions in lieu of guardrail 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-12, guardrail may be placed thereon (see Figure A-5,). Therefore place the guardrail 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 Figure A-7, $L = L_u + L_p + L_d$. For one-way traffic operations, $L_d=0$; furthermore, for the overhead sign bridge support $L_p=0$. Equation A-2 is used to find L_u because ADT is greater than 750:

(US Customary):

$$Lu = 250 - \frac{250}{25} \times 18.5 = 65\text{ft}$$

(Metric):

$$[Lu = 76 - \frac{76}{7.6} \times 5.6 = 20m]$$

Equation A-6

For the median side, L_u = 65 ft [20 m] (rounded to 75 ft [22.9 m] to conform to even lengths of guardrail) 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 guardrail.

For the right side of traffic, guardrail must be placed at the shoulder edge (Reference Figure A-5). Substituting in Equation A-2 to determine L_u:

(US Customary):

$$\mathrm{Lu} = 250 - \frac{250}{18} \times 10 = 111\mathrm{ft}$$

(Metric):

$$[\mathrm{Lu} = 76 - \frac{76}{5.5} \times 3 = 34.5\mathrm{m}]$$

Equation A-7

Using parallel placement for the entire length, L_u = 111 ft [34.5 m] (which should be rounded to 125 ft [38] to conform to even lengths of guardrail).

Using parallel placement for the entire length of guardrail for both the median and left side, placement is as shown in Figure A-13.

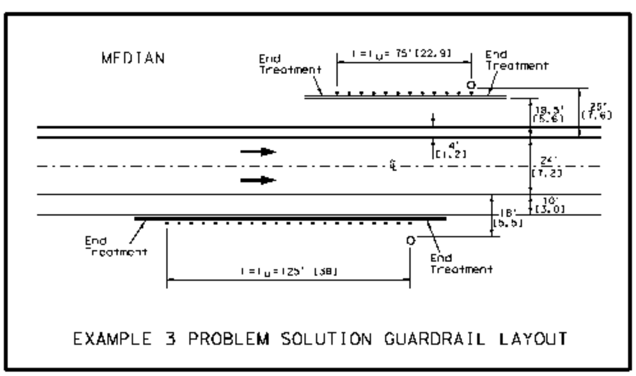


Figure A-14. Example 3 Problem Solution Guardrail Layout.

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

Contents:

Section 1 — Overview

Scope

These guidelines apply to construction zone work where continuous pavement edges or drop-offs exist parallel and adjacent to a lane used for traffic. These guidelines do not apply to short term operations. The *Texas Manual on Uniform Traffic Control Devices* (*TMUTCD*) defines short term operations as daytime work from one to twelve hours.

These guidelines do not constitute a rigid standard or policy; rather, they are guidance to be used in conjunction with engineering judgment.

Types of Treatment

Treatment may consist of either or both of the following:

- warning devices (such as signs or channelizing devices)
- 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

Factor	Definition	Notes
edge condition	slope of the drop-off	For more information, see "Edge Condition" subheading below.
Lateral clearance	distance from the edge of the travel lane to the edge condition	See Figure B-1 for description.
edge height	depth of the drop-off	See Figure B-1 for description.

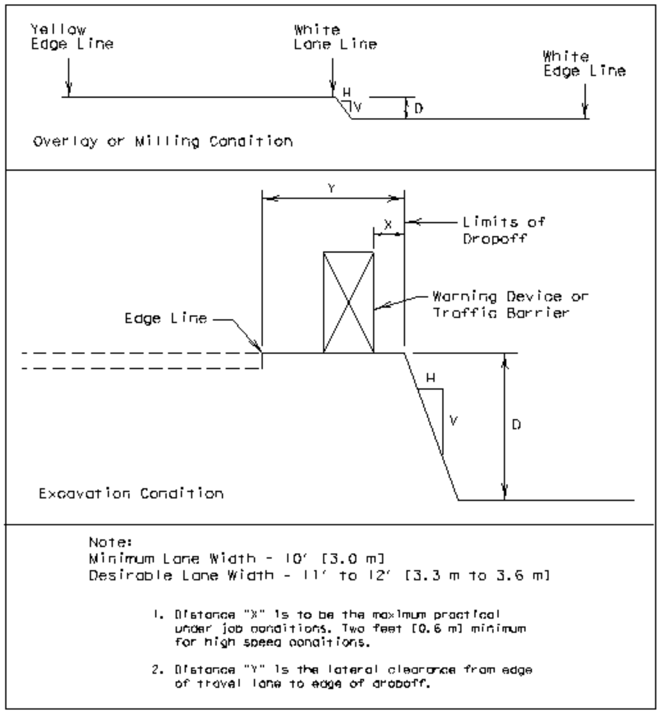


Figure B-1. Definition of Terms.

In addition to the factors considered in the guidelines, each construction zone drop-off situation should be analyzed individually, taking into account other variables, such as:

- traffic mix
- posted speed in the construction zone
- horizontal curvature
- 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.

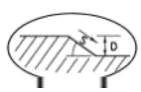
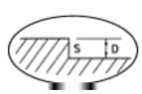
If distance *Y* (as described in Figure B-1) must be less than 3 feet [0.9 m], use of positive barrier may not be feasible. In such a case, if a positive barrier is needed (according to Figure B-2), then consider one of the following:

- moving the lane of travel laterally to provide the needed space
- 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.

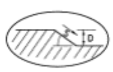
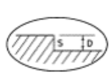
Edge Condition Types

Condition Type & Description	Notes
Edge Condition I  *S* = 3:1 or flatter slope rate (*H:V*)	Most vehicles are able to traverse an edge condition with a slope rate of 3 to 1 (horizontal to vertical) or flatter. The slope must be constructed with a compacted material capable of supporting vehicles.
Edge Condition II  *S* = 2.99:1 to 1:1 slope rate (*H:V*)	Most vehicles are able to traverse an edge condition with a slope between 2.99 to 1 and 1 to 1 (horizontal to vertical) as long as *D* does not exceed 5 inches [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.
Edge Condition III  *S* is steeper than 1:1 slope rate (*H:V*)	Slopes steeper than 1 to 1 (horizontal to vertical) where *D* is greater than 2 inches [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. NOTE: Milling or overlay operations that result in Edge Condition III should not be in place without appropriate warning treatments, and these conditions should not be left in place for extended periods of time.

Guidelines for Treatment

The 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

Edge Condition	Lateral Clearance	Edge Height	Usual Treatment (See Note 3)
I  (slope is 3:1 or flatter)	30 ft. [9 m]	0 to 1 in. [0 to 25 mm]	no treatment
-	--	>1 to 2 in. [>25 to 50 mm]	CW 8-11 signs
-	--	> 2 in. [> 50mm]	CW 8-9a or CW 8-11 signs plus channelizing devices
-	> 30 ft. [> 9 m]	Any height	no treatment
II  (slope is between 2.99:1 and 1:1)	20 ft. [6 m]	0 to 1 in. [0 to 25 mm]	no treatment
-	-	>1 to 2 in. [>25 to 50 mm]	CW 8-11 signs
-	-	>2 to 5 in. [>50 to 125 mm]	CW 8-9a or CW 8-11 signs plus channelizing devices
-	-	>5 to 24 in. [>125 to 600 mm]	CW 8-9a or CW 8-11 signs plus drums (see Note 1)
-	-	> 24 in. [> 600 mm]	Check indications for positive barrier (See Note 2)
-	> 20 ft. but 30 ft. [> 6 m but 9 m]	0 to 1 in. [0 to 25 mm]	no treatment
-	-	>1 to 2 in. [>25 to 50 mm]	CW 8-11 signs
-	-	> 2 in. [> 50mm]	CW 8-9a or CW 8-11 signs plus channelizing devices
-	> 30 ft. [> 9 m]	Any height	no treatment
III  (slope is steeper than 1:1)	20 ft. [6 m]	0 to 1 in. [0 to 25 mm]	no treatment
-	-	>1 to 2 in. [>25 to 50 mm]	CW 8-11 signs
-	-	>2 to 24 in. [>50 to 600 mm]	CW 8-9a or CW 8-11 signs plus drums (see Note 1)

Treatment Guidelines for Pavement Drop-offs in Construction Work Zones

Edge Condition	Lateral Clearance	Edge Height	Usual Treatment (See Note 3)
-	-	> 24 in. [> 600 mm]	Check indications for positive barrier (See Note 2)
-	> 20 ft. but 30 ft. [> 6 m but 9 m]	0 to 1 in. [0 to 25 mm]	no treatment
-	-	>1 to 2 in. [>25 to 50 mm]	CW 8-11 signs
-	-	> 2 in. [> 50mm]	CW 8-9a or CW 8-11 signs plus channelizing devices
-	> 30 ft. [>9 m]	Any height	no treatment

NOTE: Where restricted space precludes the use of drums, use channelizing devices. An edge fill may be provided to change the edge slope to that of the preferable Edge Condition I.

NOTE: Check indications for positive barrier (Figure B-2). Where positive barrier is not indicated, CW 8-9a or CW 8-11 signs plus drums may be used (with Note 1 also applying) after consideration of other applicable factors.

NOTE: Channelizing devices for the purpose of dropoff conditions are defined as: vertical panels, edge-line channelizers, or drums.

Use of Positive Barriers

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.

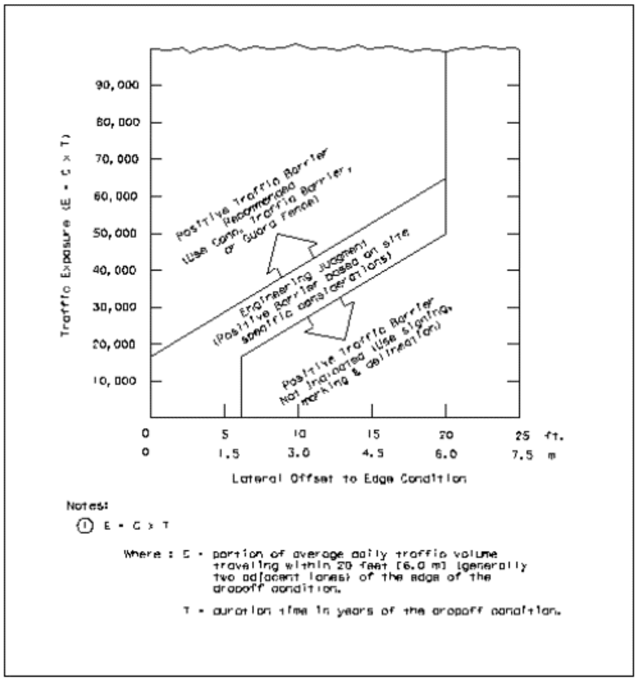


Figure B-2. Conditions Indicating Use of Positive Barrier.

Appendix C — Driveway Design Guidelines

Contents:

Section 1 — Purpose

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.

Additional information can also be found in *Regulations for Access Driveways to State Highways* for permitting guidelines and the TxDOT *Access Management Manual* 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 from the development served. 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. **Delivery 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**: Facility for entry and/or exit such as driveway, street, road, or highway that connects to the road under the jurisdiction of the department or municipality.
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 7.5 m [25 ft].
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 length of driveway as measured from the right-of-way line to the first on-site intersection or parking stall.
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 various 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 that cross sidewalks are located in a developing area where pedestrian traffic can be expected, should be designed to maintain an accessible route that is at least four feet wide across the driveway.
6. One-way driveways should have a minimum throat length of 50 feet (15 m) and preferably 75 feet (23 m).

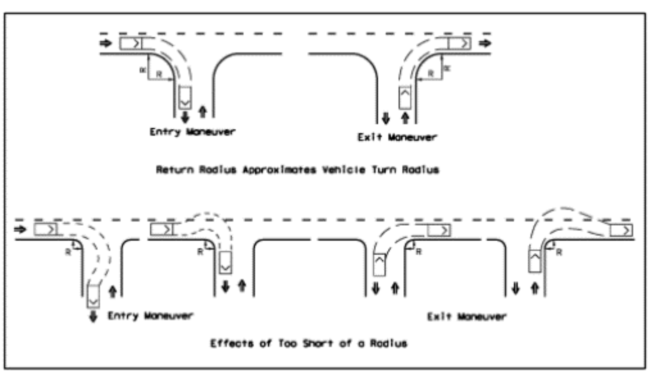


Figure C-1. Effects of Return Radius on the Right-Turn Maneuver

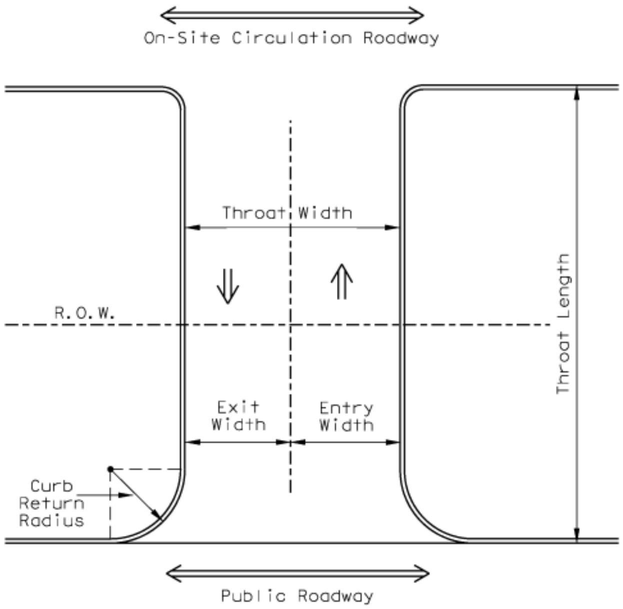


Figure C-2. Driveway Design Elements

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 C-1. Design Criteria for Private Residential Driveways

Radius	**Throat Width**		**Radius**	**Throat Width**	
US Customary Units			**Metric Units**		
(ft.)	Standard (ft.)	Maximum (ft.)	(m)	Standard (m)	Maximum (m)
15 1.	14	24	4.5	4.2	7.2

Table C-1. Design Criteria for Private Residential Driveways

Radius	Throat Width	Radius	Throat Width
1. Reference Regulations for Access Driveways to State Highways for suggested minimum values.			

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.

Table C-2. Designs for Two-Way Commercial Driveways

	US Customary Units		Metric Units	
Condition	Radius (R) (ft)	Throat Width (W) (ft)	Radius (R) (m)	Throat Width (W) (m)
One entry lane and one exit lane, fewer than 4 large vehicles per hour (see Fig. C-3)	25	28	7.5	8.4
One entry lane and one exit lane, 4 or more SU vehicles[3] per day (see Fig. C-3)	30	30	9.0	9.0
One entry lane and two exit lanes, without divider (see Fig. C-4)	25	40	7.5	12.0
One entry lane and two exit lanes, with divider (see Fig. C-5)	25	44[(1)]-50[(2)]	7.5	13.2[(1)]-15.0[(2)]
Two entry lanes and two exit lanes, with divider (see Fig. C-6)	25	56[(1)]-62[(2)]	7.5	16.8[(1)]-18.9[(2)]
(1)4 ft. [1.2 m] wide divider, face-to-face of curbs (2)10 ft. [3.0 m] wide divider, face-to-face of curbs (3)Driveway designs for larger vehicles will be considered on a case by case basis				

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.

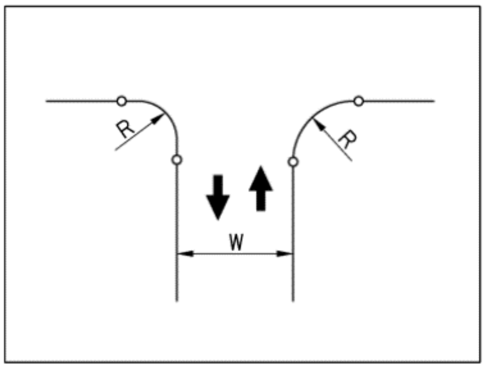


Figure C-3. One Entry Lane/One Exit Lane

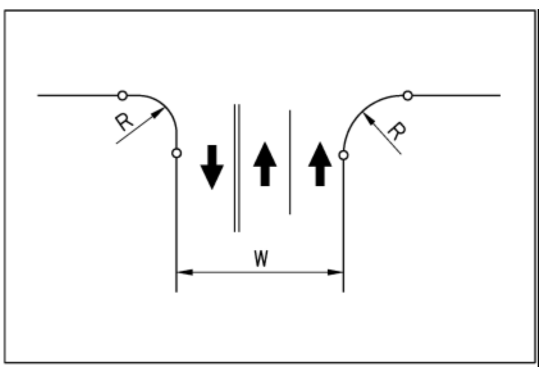


Figure C-4. One Entry Lane/Two Exit Lanes (Without a Divider)

See Table C-2 for Suggested Dimensions Based on Conditions.

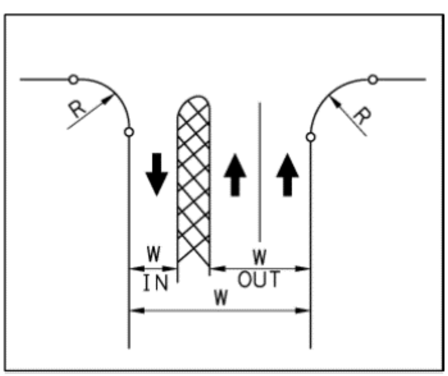


Figure C-5. One Entry Lane/Two Exit Lanes (With a Divider)

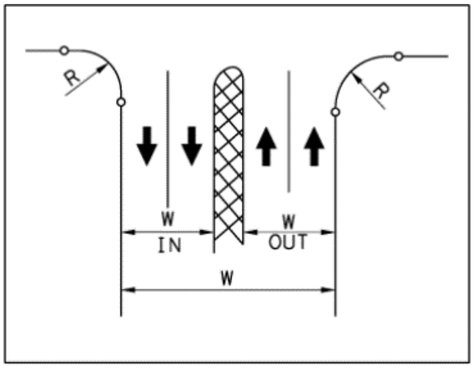


Figure C-6. Two Entry Lanes/Two Exit Lanes (With a Divider)

See Table C-2 for Suggested Dimensions Based on Conditions.

Divided Driveways

A raised or depressed separation between the entry and exit sides of a divided driveway needs to be visible to drivers. Suggested treatments and divider sizes are shown in Table C-3:

Table C-3. Dimensions for Dividers in the Driveway Throat to Separate Entry and Exit Sides of the Driveway

Treatment	Width	Length
Slightly raised(1) (4in [100 mm]) with contrasting surface(1)	4 –15 ft [1.2 – 4.5 m]	20 ft [6.0 m]
(1)For Rural - Rounded edges, 30° to 45° slope. (See Figure C-7)		

Figure C-7 illustrates a slightly raised divider (height 4 inches [100 mm]).

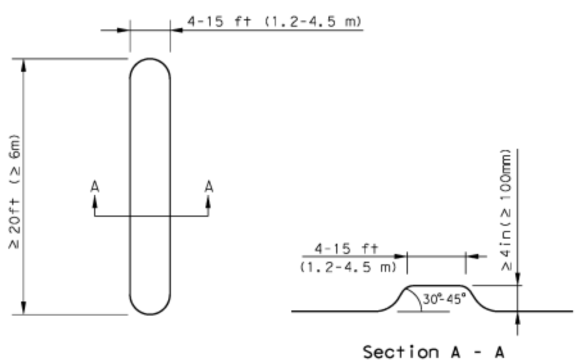


Figure C-7. Illustration of Slightly Raised Divider

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].

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.
   - 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

Driveway	Change in Grade (A)(1)
Private Residential Driveways	10%
All Other Driveways	8%
(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].

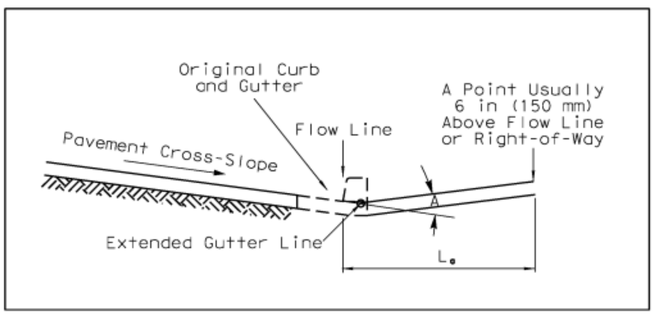


Figure C-8. Suggested Dimensions to Achieve an Appropriate Vertical Curve

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

Change in Grade, A				
	Crests		Sags	
	Des.	Min.	Des.	Min.
	ft (m)	ft (m)	ft (m)	ft (m)
4-5%	5 (1.5)	3 (0.9)	7 (2.1)	4 (1.2)
6-7%	6 (1.8)	4 (1.2)	8 (2.4)	5 (1.5)
8-10%	8 (2.4)	5 (1.5)	10 (3.0)	7 (2.1)
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

	Crest		**Sag**	
Change in Grade A	Private Residential Driveways	Other Driveways	Private Residential Driveways	Other Driveways
	ft (m)	ft (m)	ft (m)	ft (m)
4-5%	2 (0.6)	5 (1.5)	3 (0.9)	6 (1.8)
6-7%	3 (0.9)	5 (1.5)	5 (1.5)	7 (2.1)
8-10%	4 (1.2)	6 (1.8)	6 (1.8)	8 (2.4)

Profiles on Curb and Gutter Sections

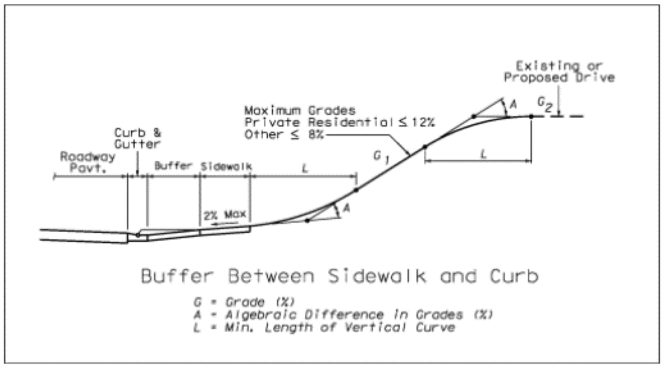


Figure C-9. Roadway with Curb and Gutter, Driveway Profiles on an Upgrade

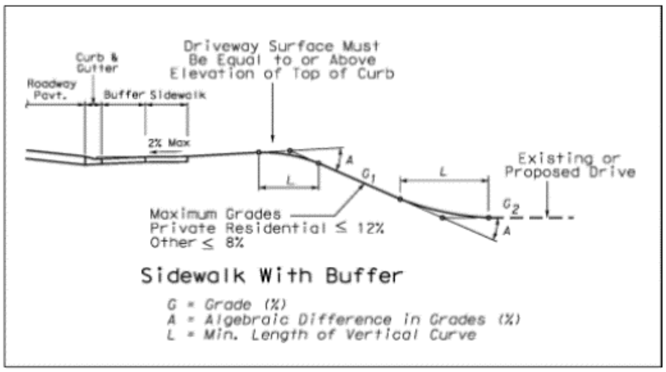


Figure C-10. Roadway with Curb and Gutter, Driveway Profiles on a Downgrade

See Tables C-5 and C-6 for lengths of vertical curves.

Profiles with Drainage Ditch

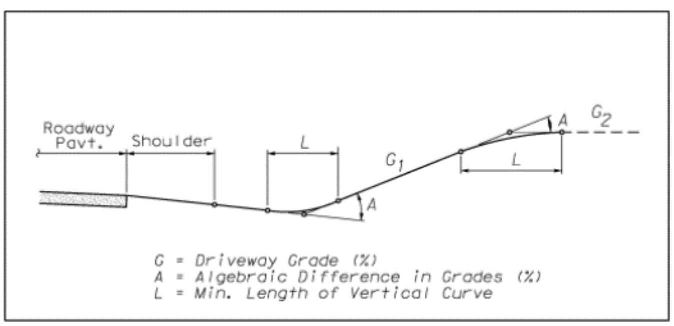


Figure C-11. Driveway Profiles on Roadway with Drainage Ditch

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^{o}

Commercial Driveway: 75^{o}; 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^{o}.

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^{o}) and ninety degrees (90^{o}). Sixty degrees (60^{o}) 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 5 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.)

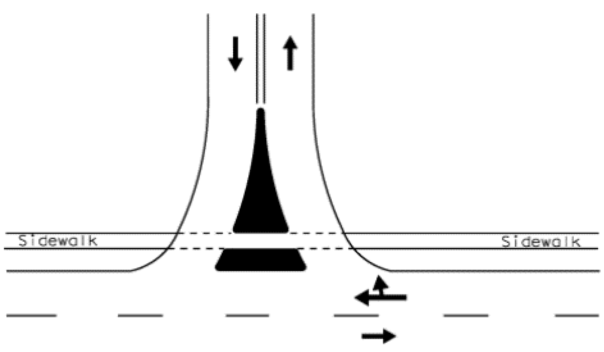


Figure C-12. Channelizing Island to Provide Pedestrian Refuge

- 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.

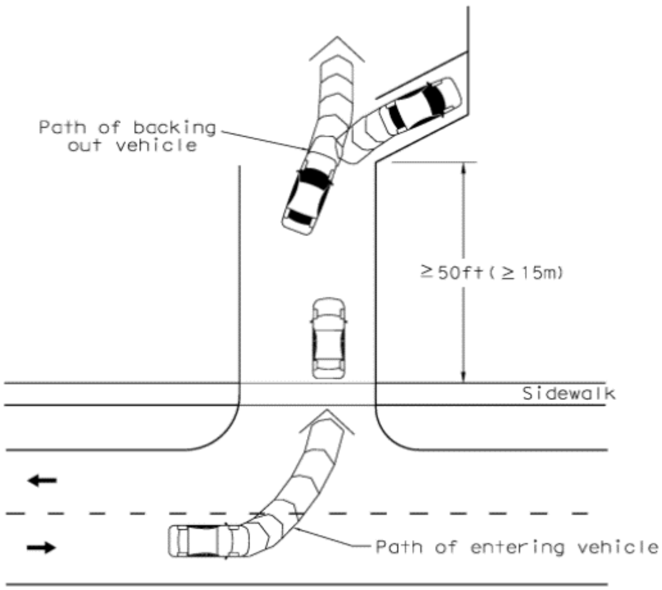


Figure C-13. Throat Length is of Sufficient Lenght to Allow Entering Vehicle to Clear the Through Traffic Lane

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.

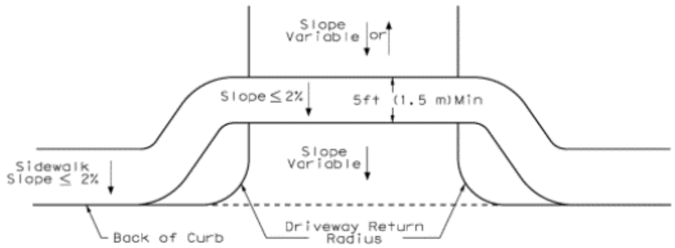


Figure C-14. Illustration of a "Walk-Around" Design

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

1. A Policy on the Geometric Design of Highways and Streets, American Association of Highway and Transportation Officials, 2001.
2. Transportation and Land Development, Institute of Transportation Engineers, 2002.
3. Access Management Manual, Transportation Research Board, 2003.
4. Draft Guidelines for Accessible Public Rights-of-Way, November 25, 2003.
5. R. J. Jaeger, Guidelines for the Investigation and Remediation of Potentially Hazardous Bicycle and Pedestrian Locations, Traffic Engineering and Safety Management Branch, North Carolina Department of Transportation, August 2003.
6. Charles V. Zegeer, Cara Seiderman, Peter Lagerway, Mike Cynecki, Michael Ronkin and Robert Schneider, "Pedestrian Facilities Users Guide – Providing Safety and Mobility," Publication No. FHWA No. FHWA-RD-01-102, Federal Highway Administration, US Department of Transportation, March 2002