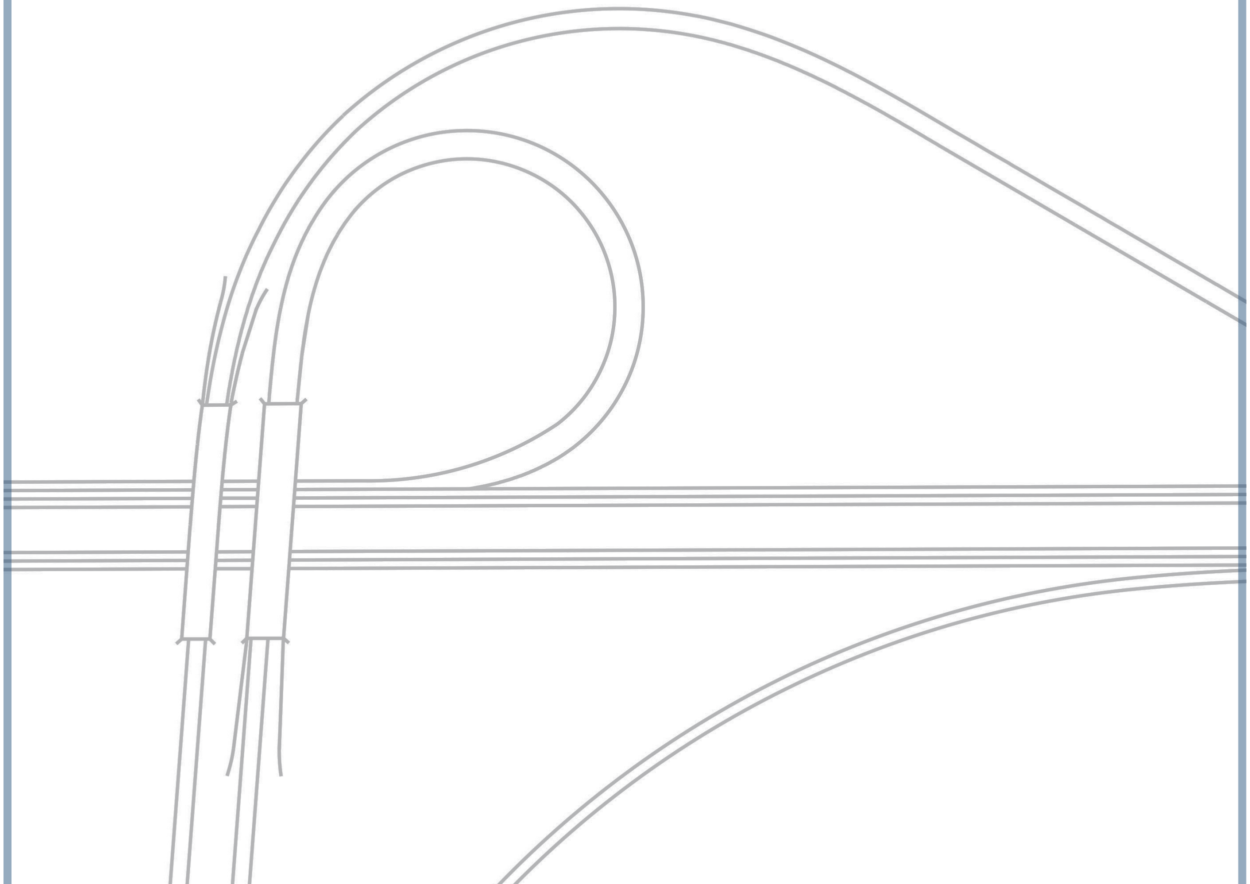
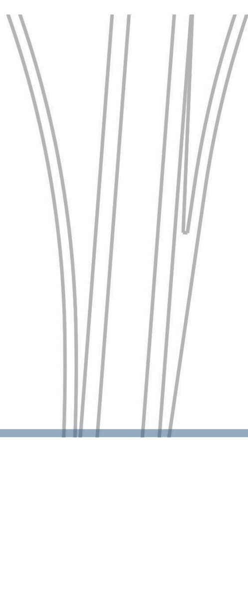
2020 ROADWAY Design Manual

MDOT
MISSISSIPPI DEPARTMENT OF TRANSPORTATION



CHAPTER 1

General Design Guidance



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Chapter 1
GENERAL DESIGN GUIDANCE

The principal purpose of this manual is to guide designers in the use of design criteria and procedures that will consistently result in quality Roadway Design Plans. Updates and/or modifications to the manual may be warranted throughout its existence. Updates and/or modifications that are considered minor, such as typographical changes or other changes in details that do not require great attention, may be authorized by the Roadway Design Division Engineer, or designee, after review by the Legal Division. Updates and/or modifications that are considered moderate, such as changes in design criteria or guidelines from other publications as listed in Section 1-4.0, will be preceded by design memos authorized by the Roadway Design Division Engineer after review by the Legal Division. Updates and/or modifications having a significant impact are considered major, such as a significant rewrite of the manual, may only be authorized by the Executive Director after review by the Legal Division. Design memos may be accessed electronically via website or links until such time that a formal update can be incorporated into the manual itself.

Roadway Design Division holds two basic responsibilities:

- to provide the design for each roadway project in accordance with the applicable criteria
- to combine all design elements (e.g., roadway, bridge, signing) into a complete set of Contract Plans to be used for construction of the project.

The geometric design of a roadway typically addresses the elements of location, functional classification, horizontal and vertical alignment, cross section, intersections, control of access, pedestrians, and bicyclists. Roadside safety, drainage, and traffic control are also incorporated into roadway design. The use of these elements in proper combination is the task of the roadway designer.

The Department has established new construction/reconstruction geometric design criteria for most roadway elements (e.g., lane and shoulder widths, slopes, grades, curvature, clearances), which can be found in Section 2-9.0 (rural roadways) and Section 14-2.0 (urban roadways).

The Department has also established different sets of geometric design criteria known as Resurfacing, Restoration, and Rehabilitation (3R) criteria, as well as Preventive Maintenance (1R) criteria as presented in Chapter 12, “Existing Roadways”, which are typically used on existing roadways where it is not feasible to use the criteria for new construction/reconstruction projects.

This manual is intended to only apply to proposed roadway projects as discussed in Section 2-7.0. Nothing herein is intended to be construed as a representation that all existing roadways currently meet the guidelines and/or criteria of this manual.

1-1.0 DESIGN FLEXIBILITY

Design flexibility should always be considered because each project has a specific purpose and need, but each project also has specific context and constraints (e.g., environmental, right of way impacts). No project is exactly like another; therefore, no single set of design criteria can be applicable to or meet the needs of all, or even most, projects.

The purpose of a project may be to specifically address a section of the roadway that needs repair or is not performing satisfactorily. If some other existing aspects of the roadway that are not contributing factors to the specific purpose and need of a project, either within or adjacent to the project limits, do not fully comply with the geometric design criteria presented in Chapters 2 through 14 or any other relevant publications, there is no reason to change those existing aspects of the roadway. Simply applying geometric design criteria without regard to the context of the project would be a potential waste of limited funds available for transportation improvements that could be better spent addressing identified needs on other roadways.

1-2.0 DEFINITIONS

1-2.01 Qualifying Words

1. Mandatory condition – Designers are strongly encouraged to follow the criteria and presented in this context unless reasonable justification exists not to do so. The application of this wording is as follows:

    a. Language that is used in reference to geometric design criteria for freeways, which is consistent with the language in the AASHTO publication *A Policy on Design Standards – Interstate System*

    b. Language that is also used in reference to the design of pedestrian facilities, which is consistent with the language used in the United States Access Board publication *Public Right-of-Way Accessibility Guidelines (PROWAG)*

    c. Examples of words that indicate a mandatory condition are "shall", "require", "will", and "must".

    d. Where the criteria presented in this context will not be met, approval by the Chief Engineer will be required. See Section 2-10.0.

2. Advisory condition – Designers are encouraged to follow the criteria and guidance presented in this context unless reasonable justification exists not to do so. Any such criteria and guidance is representative of generally accepted limits, but is not necessarily absolute.

    a. Examples of words that indicate an advisory condition include "should" and "recommend".

    b. Examples of limits that indicate an advisory condition include "minimum" and "maximum".

    c. Where the criteria presented in this context will not be met, approval by the Chief Engineer may be required. See Section 2-10.0.

    d. In this manual, the word "should" used in combination with a permissive condition (e.g., "should desirably", "should preferably", "should, if feasible", "should be considered") is not intended to indicate an advisory condition, but rather a permissive condition.

3. Permissive condition – Designers should make every reasonable effort to meet the criteria and guidance presented in this context, and should only use a lesser design after due consideration of the better design.

    a. Examples of words that indicate a permissive condition include may, could, can, suggest, consider, desirable, and preferred.

    b. Approval by the Chief Engineer is not required where the criteria presented in this context will not be met.

4. Warranted, Justified – Indicating that some well-accepted threshold or set of conditions has been met. As used in this manual, warranted or justified may apply to either objective or subjective evaluations. Once the warranting threshold has been met, it is an indication that the design treatment should be considered and evaluated, but not that the design treatment is automatically required.

5. Guidelines, Guidance – Indicating a design value that establishes an approximate threshold that should be met if considered feasible.

6. Criteria – A term typically used to apply to design values, usually with no suggestion on the criticality of the design value. This manual frequently uses criteria to refer to the design values presented.

7. Typical – Indicating a design practice that is often used in application and is likely to be the best treatment at a given site.

8. Acceptable – Design criteria that do not meet desirable values but are considered reasonable and safe for design purposes.

9. Efficient – Indicating that a function has been performed in the best possible manner with the least waste of time, effort, and cost. This term replaces the use of the word "effective" that is used in various other design resources.

1-2.02 Acronyms

1. AASHTO – American Association of State Highway and Transportation Officials
2. *ADA – Americans with Disabilities Act*
3. *ADAAG – Americans with Disabilities Act Accessibility Guidelines*
4. FHWA – Federal Highway Administration
5. *MASH – Manual for Assessing Safety Hardware*
6. MDOT – Mississippi Department of Transportation
7. *MUTCD – Manual on Uniform Traffic Control Devices*
8. NCHRP – National Cooperative Highway Research Program
9. NHS – National Highway System
10. *PROWAG – Public Right of Way Accessibility Guidelines*
11. TRB – Transportation Research Board

1-3.0 SAFETY CONSIDERATIONS

The Department has established geometric design criteria for most roadway elements (e.g., lane and shoulder widths, slopes, grades, curvature, clearances). Tables of geometric design values based on roadway functional classification and urban/rural location are presented in Chapters 2, 12, and 14.

1-3.01 Driver Expectancy

Driver expectancy relates to the likelihood that a driver will successfully respond to common situations. Design features of roadways should be similar to create expectancies related to common geometric, operational, and route characteristics. Drivers should be able to perceive information and safely modify the speed and accuracy of their responses.

Chapter 2, "Design Controls and Criteria", of AASHTO's *A Policy on Geometric Design of Highways and Streets* provides guidance on driver performance and other human factors.

1-4.0 SOURCES OF INFORMATION

This manual is a principal source of information for designers. Other information sources are discussed in the following subsections.

1-4.01 Technical Assistance

Supplementary expertise may be provided by specialists in other fields (e.g., environmental considerations, pavement design, bridge design, construction). Therefore, the Roadway Design Division exchanges information with other Divisions of the Department during the design process.

1-4.02 Department Publications

Designers should frequently refer to other resources for any updates to design procedures. Such resources include, but are not limited to, the following materials found on the Department internet or intranet sites:

1. Design Memos – Designers will receive memos defining new or revised procedures. Periodically, the design memos will be incorporated into revisions to the *Roadway Design Manual*.

2. MDOT's *Roadway Design Standard Drawings* – The Mississippi Department of Transportation's *Roadway Design Standard Drawings* are drawings of standardized details that are common to all projects.

3. MDOT's *Access Management Manual* – The Mississippi Department of Transportation's *Access Management Manual* contains criteria related to the control of access along roadways in the state.

4. Special Design Sheets – Typical detail drawings not covered in the *Standard Drawings*.

5. CADD Manual – This manual establishes the computer-aided drafting and design standards and covers the tools, techniques, applications and procedures that are used to produce a complete project using MicroStation.

6. *Survey Manual* – The Mississippi Department of Transportation's *Survey Manual* defines the minimum specifications and procedures when performing surveys for the Department.

7. *Standard Specifications* – The *Mississippi Standard Specifications for Road and Bridge Construction* describes the various construction work items and the Department's relationship with contractors.

8. Special Provisions – A file of Special Provisions is maintained to identify current amendments (e.g., additions, deletions, revisions) to the *Standard Specifications*.

9. *Standard Operating Procedures (SOP)* – Official Department rules, guidelines, and operating procedures are listed for use in all functions.

10. Department Rules – Because the public is affected by these regulations, they are listed with the Secretary of State in compliance with the Administrative Procedures Act (APA).

1-4.03 AASHTO Publications

Department design criteria and policies adhere closely to policies established by AASHTO. This manual tailors the AASHTO design criteria specifically for application in Mississippi, and addresses design issues not covered in the national publications. The intent of this manual is to provide a single source document that answers the vast majority of questions for the designer. However, the national publications provide much of the theory and background for geometric design and roadside safety criteria. Therefore, designers should be familiar with the latest editions of the AASHTO publications listed below and review the documents for explanations of the concepts that serve as the basis for much of the design criteria:

1. *A Policy on Geometric Design of Highways and Streets (Green Book)*
2. *A Policy on Design Standards – Interstate System*
3. *LRFD Bridge Design Specifications*
4. *Roadside Design Guide*
5. *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*
6. *Highway Safety Manual* (HSM)
7. *Guide for the Development of Bicycle Facilities*
8. *Guide for the Planning, Design, and Operation of Pedestrian Facilities*
9. *Guidelines for Geometric Design of Low-Volume Roads*
10. *Manual for Assessing Safety Hardware* (*MASH*)
11. *A Guide for Achieving Flexibility in Highway Design*
12. *Guide for Geometric Design of Transit Facilities on Highways and Streets*

Many other AASHTO publications are available to provide authoritative guides and policies in specific areas (e.g., pavement design, drainage design, landscaping, rest areas, lighting, utilities).

1-4.04 Other Publications

This manual is consistent with applicable FHWA requirements related to geometric design and roadside safety. Many of these directives have been formally published as FHWA Notices, FHWA Orders, FHWA Technical Advisories, FHWA policy memoranda, FHWA guidance memoranda, etc., which can be found on the FHWA internet site.

The *MUTCD* presents information on the type, location, and design of traffic control devices approved by FHWA, AASHTO, Institute of Transportation Engineers, and American Traffic Safety Services Association for roadway use. The *MUTCD* is continually updated, and the designer should be aware of these revisions.

Designers should also consider the following list of publications:

- NACTO's *Urban Street Design Guide*
- ITE's *Designing Walkable Urban Thoroughfares Guide*
- FHWA's *Flexibility in Highway Design*
- *U.S. Access Board's Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way*

1-4.05 TRB and NCHRP Reports

The NCHRP reports and various TRB publications, including the *Highway Capacity Manual*, cover the entire field of roadway and traffic engineering. TRB publications before 1974 are identified as Highway Research Board (HRB) reports.

Copies of these reports are available on the TRB website and in Roadway Design Division's library.

1-4.06 Supplier Publications

Information on special design is often available through the suppliers of various construction materials, which may be useful in the design of drainage facilities, retaining walls, traffic barriers, traffic control devices, etc. Upon request, engineers from the Materials Division and the Construction Division may furnish the designer with information related to specific design.

1-5.0 PROJECT FILE

Each project should have an electronic project file associated with it that contains miscellaneous forms, reports, and correspondence. The designer and/or Roadway Design Section Engineer should ensure that all such documents are placed in the appropriate folder in the Department's electronic file storage application. The following are some of the items that are typically included in the project file for Department projects:

1. Location Committee Report – A report that documents the scope of work for the project and the justification for the recommended alternative versus other alternatives that were eliminated during discussions at the location committee meeting. Also included in the report are items such as the RWD-600, ENV-160, and cost estimate(s).

2. Form RWD-200 "Field Notes and Sketches" – A form that documents the inventory of all field data survey notes, right of way notes, incidental notes, as well as miscellaneous maps, plans, and sketches.

3. Approved Pavement Design – Chapter 13, "Pavement Design", discusses the procedures for determining the pavement design for each project.

4. Soils Data – The District prepares and submits this report on the testing of soil samples and a graphical sketch of the soil profile throughout the proposed project.

5. Form RWD-600 "Project Design Data" – A form that documents basic design data instructions for roadway construction projects as prepared by the Roadway Design Section Engineer and approved by the Roadway Design Division Engineer and District Construction Engineer.

6. Form ENV-160 "Environmental Class of Determination and Gold Commitment Sheet(s) – The ENV-160 form is prepared by the Environmental Division and is used to make a preliminary determination on the environmental class of action (EIS, EA, or CE). This form also serves as the final Environmental Document for CE (Categorical Exclusion) projects. Environmental mitigation measures are outlined on the Gold Commitment Sheet(s). Section 2-7.01 provides more information on the types of environmental actions.

7. Field Inspection/Office Review Reports – Each of these plan reviews is followed by the submittal of a report that documents the plan changes and discussions that were held during the meeting. Each report should also include the RWD-600, updated cost estimate, and the Environmental Gold Commitment Sheet.

8. Approved Design Exception/Variance – Chapter 2, "Basic Design Controls", discusses the Design Exception/Variance process that should be followed when design criteria cannot be met.

9. Checklists – The Department has developed several quality control checklists to assist the designer throughout the process of preparing right of way and construction plans. The designer should complete the appropriate checklist for each milestone during plan development. These checklists can be found on the Department's website.

10. Cost Estimates – Cost estimates should be updated throughout the plan development of a project.

11. Calculations – The designer should keep a neat and organized copy of all quantity and design calculations.

12. Miscellaneous Reports – Each project includes reports prepared by others, which may include, but are not limited to, geotechnical report, hydraulic analysis, traffic analysis, and minutes from meetings with consultants.

Upon completion of the project, the project file is retained by the Department.

1-6.0 PROJECT REPORTS

Reports should be submitted to the Roadway Design Division Engineer, with copies provided to Central Files, the project file, and heads of all other applicable units and/or individuals (e.g., FHWA, District, Construction Division, Traffic Engineering Division, Geotechnical Branch, Bridge Division, Hydraulics Branch, Right of Way Division, Programming Manager).

1-7.0 QUANTITY ESTIMATES

The designer should use the units of measurement provided in the *Standard Specifications.* Some common units of measurement are shown below in Table 1-7-A, along with uniform criteria for degree of accuracy and rounding of computations.

Table 1-7-A
UNIT ROUNDING OF PRELIMINARY QUANTITY ESTIMATES

Measurement	Unit	Description	Nearest Unit: Estimated Quantity Sheets (Recap)	Nearest Unit: *Summary of Quantities
Bale	EACH	all items	1 bale	1 bale
Cubic Yard	CY	all items	1 CY	1 CY
Each	unit	all items	1 unit	1 unit
Acre	ACRE	all items	0.1 ACRE	1 ACRE
Pounds	LBS	all items	1 LB	1 LB
Mile	MI	all items	0.001 MI	1 MI
Gallon	GAL	all items	1 GAL	1 GAL
1000 gallons	KGAL	all items	0.1-1000 GAL	1-1000 GAL
Linear Foot	LF	all items except: pipe culvert concrete median barrier guardrail	1 LF 8-ft increments 10-ft increments 12.5-ft increments	1 LF 1 LF 1 LF 1 LF
Ton	TON	all items	1 TON	1 TON
Square Foot	SF	all items	0.1 SF	1 SF
Square Yard	SY	all items	1 SY	1 SY
Thousand	1000	all items	0.1-1000	1 thousand

**Note:* *When quantities are smaller than one unit, the quantity should be rounded to one unit.*

The Summary of Quantities in the plans should include all applicable pay items for the contract. The pay items should be consistent with the current pay item list located on the Contract Administration Division website.

Tabulations of estimated quantities for some pay items are included in the plans on the Estimated Quantities sheets.

1-8.0 CONSTRUCTION COST ESTIMATES

As a project progresses, construction cost estimates are typically updated throughout the various stages of project development to ensure that sufficient funds are available for construction. This section briefly discusses the various project cost estimates and who is responsible for their preparation.

1. Programming Estimate – The initial project cost estimate, which is prepared prior to any plan details being available, should be developed by the District using broad units of cost (e.g., cost per mile, cost per square yard, cost per intersection), and by reviewing similar recent projects in the area. The District may request assistance from other Divisions (e.g., Right of Way, Bridge Design, Traffic Engineering, Environmental, Roadway Design) in determining the program cost estimate.

2. Location Committee Estimate – After the Location Committee Meeting is held and the scope of the project is determined, the Location Committee Report is issued, which will include an updated cost estimate for the recommended alternative. This cost estimate will be based on historical cost data from similar projects furnished by the Planning Division. The Roadway Design Section Engineer should send the updated cost estimate to the Programming Manager.

3. Field Inspection Estimate – The Roadway Design Section Engineer should prepare a more detailed construction cost estimate following the Field Inspection. This estimate is typically prepared using proposed construction as shown in the Field Inspection Plans and historical cost data from similar projects furnished by the Planning Division. The Roadway Design Section Engineer should include the updated cost estimate with the Field Inspection Report. The Programming Manager should be included in the distribution of the report.

4. Office Review Estimate – The cost estimate should be updated using actual quantities from the Office Review Plans and included in the Office Review Report. The Programming Manager should be included in the distribution of the report.

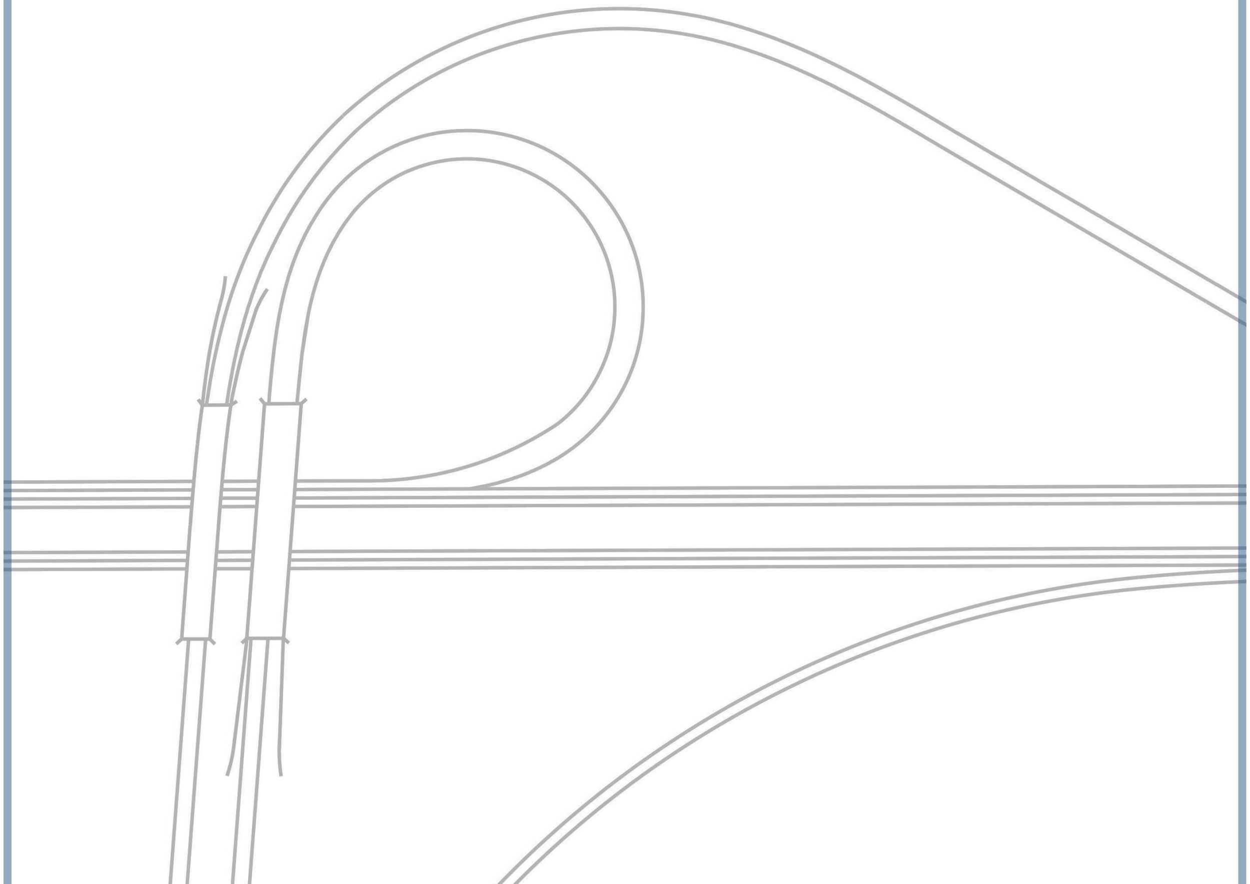
5. Revised Estimates Due to Scope of Work Changes – If at any point during the design process there are deviations from a project's scope of work, significant changes to the design, or significant increases of construction costs, the Roadway Design Section Engineer should update the cost estimate accordingly and send it to the Programming Manager.

A 15% adjustment for engineering and contingencies should be added to the construction cost estimate (20% if estimate is over $20 million).

The estimate should also be adjusted for expected inflation between the estimated and actual project letting date.

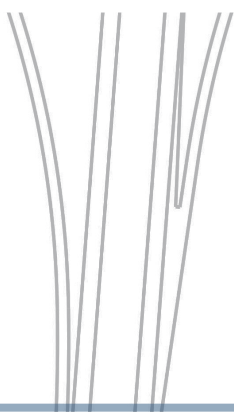
1-9.0 ROADWAY DESIGN PLANS

The following chapters provide criteria and procedures for the geometric design of a roadway facility. The appropriate design should be incorporated into the Contract Plans in a manner that can be clearly understood by contractors, material suppliers, and Department personnel during construction of the project. To ensure an accurate and consistent interpretation of the designer's intentions, the plans should include consistent format, content, and placement of information. See Chapter 15 "Roadway Design Plan Assembly" for more information.



CHAPTER 2

Basic Design Controls



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Chapter 2
BASIC DESIGN CONTROLS

Several overall roadway design controls provide the framework that establishes the geometric characteristics of a proposed roadway facility. These controls may have a significant impact on the operational efficiency, safety, and cost efficiency of the roadway investment. This chapter addresses the following basic design controls:

1. roadway systems
2. roadway speed
3. roadway capacity
4. control of access
5. project scope of work

Other topics addressed in this chapter include:

1. context sensitive solutions
2. pedestrian and bicycle considerations
3. deviations from geometric design criteria
4. value engineering

This chapter also provides the new construction/reconstruction geometric design criteria applicable to rural roadways. Geometric design criteria for projects on existing roadways and urban roadways involve additional considerations. Therefore, Chapter 12, "Existing Roadways", provides the geometric design criteria for projects on existing rural and urban roadways, and Chapter 14, "Geometric Design of Urban Roadways", provides new construction/reconstruction criteria for projects on urban roadways.

AASHTO is the recognized authority on roadway design policies and criteria. Since 1938, AASHTO has been developing and publishing design policies for use by highway agencies, and it continues to update policies and criteria to reflect new findings and the current state of knowledge. The AASHTO publication *A Policy on Geometric Design of Highways and Streets* (Green Book) provides the principal source of information and design criteria contained in this manual. The AASHTO criteria have been tailored to be consistent with the prevailing conditions within Mississippi.

2-1.0 ROADWAY SYSTEMS

The Mississippi roadway network has been divided into several classification systems for applying design criteria and for determining which source of funds can be used. The Planning Division maintains and updates the maps for the functional classification system, Federal-aid system, and the various jurisdictional systems.

2-1.01 <u>Functional Classification</u>

The Department's roadway design criteria are based on the functional classification concept. In this system, roadways are grouped by the character of the service they provide. The two major considerations in classifying the public roadway network are access to adjacent properties and traffic mobility. Each roadway provides varying levels of these two functions. In the functional classification scheme, the overall objective is that the roadway system, when viewed in its entirety, yields an optimum balance between its access and mobility purposes.

2-1.01.01 Arterials

Ideally, arterial roadways are characterized by limited access to abutting properties and a capacity to quickly move relatively large volumes of traffic. Rural arterials provide connections between major urban areas and provide a Level of Service (LOS) suitable for statewide or interstate travel. Urban arterials serve the major centers of activity of urbanized areas, the highest traffic volume corridors, and the longest desired trips. Rural and urban arterial systems are connected to provide continuous through movements at approximately the same LOS. For design, arterial roadways are divided into the following categories:

- Freeways – The freeway is the highest level of arterial. These facilities are characterized by full control of access, high design speeds, and a high level of driver comfort and safety. Freeways are considered a special type of roadway within the functional classification system, and separate design criteria have been developed for these facilities.

- Principal and Minor Arterials – In both rural and urban areas, principal arterials accommodate higher traffic volumes and greater trip lengths. Many of these are divided facilities, which may have partial control of access. Minor arterials provide a mix of interstate and inter-county travel service in rural areas, and provide intra-community connections in urban areas. Minor arterials, as compared to principal arterials, accommodate relatively lower travel speeds, trip lengths, and traffic volumes, but provide more access to property than the principal arterial system.

2-1.01.02 Collectors

Collector routes are characterized by an approximately even distribution of their access and mobility functions. Traffic volumes and speeds are typically lower than those of arterials. In rural areas, collectors serve intra-county travel needs and provide connections to the arterial system. In urban areas, collectors act as intermediate links between the arterial system and points of origin and destination.

2-1.01.03 Local Roads

Any public road not classified as an arterial or collector is classified as a local road. Local roads are characterized by their many points of direct access to adjacent properties and their relatively minor value in accommodating mobility. Speeds and volumes are usually low and trip lengths short.

2-1.02 Federal-Aid System

The Federal-aid system consists of those routes within Mississippi that are eligible for categorical Federal highway funds. The Department, working with local governments and in cooperation with FHWA, has designated eligible routes. Title 23 of the *United States Code* describes the applicable Federal criteria for establishing the Federal-aid system.

2-1.02.01 National Highway System

The National Highway System (NHS) is a system of roadways determined to have the greatest importance to the Nation's economy, defense, and mobility. The NHS represents approximately 4% of the total public road mileage in the United States, consisting of facilities that meet the requirements of one of the following sub-systems (note that a specific route may be on more than one sub-system):

1. Interstate – The Interstate Highway System, which retains its separate identity within the NHS

2. Other Principal Arterials – Roadways in rural and urban areas that provide access between an arterial and a major port, airport, public transportation facility, or other intermodal transportation facility

3. Strategic Highway Network – A network of highways that is important to the United States' strategic defense policy and that provides defense access, continuity, and emergency capabilities for defense purposes

4. Major Strategic Highway Network Connectors – Highways that provide access between major military installations and highways that are part of the Strategic Highway Network

5. Major Intermodal Connectors – Roadways that provide access between major intermodal facilities and the other four subsystems making up the NHS

2-1.02.02 National Highway Freight Network

Fixing America's Surface Transportation Act (FAST) requires the establishment of a National Highway Freight Network, which consists of the following components:

1. Primary Highway Freight System (PHFS)
2. Critical Rural Freight Corridors
3. Critical Urban Freight Corridors
4. Portions of the Interstate System that are not part of the PHFS

2-1.02.03 Funding Programs

Funding for transportation activities comes from many sources, which include federal, state, and local dollars. Various sources of federal transportation funding available to Mississippi include, among many others, the following: Community Development Block Grant Program, Congestion Mitigation and Air Quality Improvement Program, Congressional Earmarks/Discretionary Project Funding, Delta Regional Transportation Development Program, Highway Safety Improvement Program (HSIP), Mississippi Development Authority Funding, National Highway Performance Program (NHPP), National Highway Freight Program, Surface Transportation Block Grant Program set aside for Transportation Alternative Projects (formerly Transportation Alternatives Program), Railroad-Highway Crossings Program, and Surface Transportation Block Grant Program (STBG program projects) [formerly Surface Transportation Program (STP program projects)].

The matching ratios are generally 80% federal and 20% state for most projects. In the case of some safety-related improvements, such as the installation of traffic signals or railroad crossing protection devices, federal funds may be used to cover the entire cost. The Federal Transit Administration (FTA) will normally cover up to 80% of the cost incurred for transit capital projects, while funding guidelines for transit operating assistance require that state and local sources cover at least half of the total amount required to operate the system.

A substantial portion of Mississippi's state funding is used on projects where limited federal funds are available. Such funding includes those established by the Mississippi Legislature: the 1987 Four-Lane Program, the Gaming Roads Program, and the Vision 21 Program.

In addition to the above traditional sources, innovative funding strategies available to Mississippi include HELP Bonds, Advance Construction Program, Special Match Credit, Private Activity Bonds (PABs), Transportation Infrastructure Finance and Innovation Act of 1988 (TIFIA) Credit Program, Projects to Improve Freight Movement, and August Redistribution.

More detailed information on the above funding programs, project prioritization, and performance management can be found within the Department's current Statewide Transportation Improvement Program (STIP).

2-1.03 Jurisdictional Systems

The State of Mississippi contains approximately 75,000 miles of public roads. The network has been classified into several systems based on the responsible organization for roadway improvement, maintenance, and traffic enforcement.

2-1.03.01 State Highway System

The State Highway System consists of all highways under the jurisdiction of the Department. The system equals approximately 15%, or 11,500 miles, of all public roadways in Mississippi. In general, these routes are the most important roadways in the state, carrying the greatest traffic volumes and operating at the highest speeds.

2-1.03.02 County Road System

Mississippi's 82 county governments are responsible for all rural roadways within their boundaries that are not on the State Highway System. There are approximately 53,500 miles of county-maintained roads in Mississippi. Of the county-maintained roads, approximately 65% are paved.

2-1.03.03 Municipal System

The municipal system consists of most urban roadways within corporate limits. The extension of these routes outside the corporate limits, but still within the urban area, is generally the responsibility of the county.

2-2.0 SPEED

2-2.01 Design Speed

The design speed establishes basic criteria for certain design elements, such as horizontal and vertical curvature, superelevation, and sight distances. The following information should be evaluated when determining the project design speed:

1. Project Scope of Work – Section 2-7.0 identifies the various project scopes of work. For new construction and reconstruction, design speeds are determined from the geometric design criteria tables in Section 2-9.0 for rural roadways and Section 14-2.0 for urban roadways. These criteria should be supplemented by the discussion on design speed in this section.

   Design speeds for 3R criteria (freeways and non-freeways) are presented in Chapter 12, "Existing Roadways."

2. Range – Design speeds typically range between 30 and 70 miles per hour depending upon urban/rural location and functional classification. For design applications, the selected design speed is typically in a 10 mile per hour increment up to 50 miles per hour, although five mile per hour increments are acceptable (i.e., 35 miles per hour, 45 miles per hour).

3. Posted/Regulatory Speed Limit – For all projects, the selected design speed should equal or exceed the anticipated posted speed limit of the completed facility. This requirement recognizes the relationship between likely operating speeds and roadway design, and that the posted speed limit creates a driver expectation of safe operating speed. Section 2-2.04 provides guidance on determining posted speed limits.

4. Balance – The design speed should be a reasonable balance between topography, urban and rural character, and the functional class of the roadway. A roadway in level terrain may justify a higher design speed than one in rolling terrain, and a roadway in a rural setting may justify a higher design speed than one in an urban area.

5. Driver Expectancy – The element of driver expectancy should be considered when selecting the design speed. An example of applying this concept is the design of horizontal curves at the end of long tangent sections. The overall design speed of a roadway segment may be less than the top speeds on tangents within that segment, which suggests that horizontal curves at the end of tangents should desirably be flatter than the minimum radius allowed by the design speed to accommodate driver expectancy. This design practice provides a transition for the driver back to the overall design speed for the roadway segment.

6. Traffic Volumes – Traffic volumes may impact the recommended design speed. A roadway carrying a large volume of traffic may justify a higher design speed than a similar facility with lower traffic volumes. However, a low design speed should not be automatically assumed for a low traffic volume road where the topography is such that drivers are likely to travel at high speeds. Drivers do not adjust their speeds to the

functional classification of the roadway, but rather to the physical limitations and to traffic using the facility.

2-2.02 Running Speed

Running speed is the average speed of a vehicle over a specified section of roadway, and it is equal to the distance traveled divided by the running time (the time the vehicle is in motion). To evaluate levels of service and road user costs, the average running speed of all vehicles is often the most appropriate speed measure. Peak hour and non-peak hour average running speeds on a given roadway may vary. Therefore, when average running speed is referenced, it should be stated whether it is the peak hour, the non-peak hour, or an average for the day.

2-2.03 85th Percentile Speed

Many geometric design values are targeted to accommodate between 80% and 90% of all roadway users. The most common application of the 85th percentile value is its use as one of the factors for determining the posted speed limit along a roadway section; see Section 2-2.04. The Traffic Engineering Division is responsible for conducting field studies to determine the 85th percentile speed.

2-2.04 Posted Speed Limit

As discussed in Section 2-2.01, the selected design speed should equal or exceed the anticipated posted speed limit of the completed facility. The Traffic Engineering Division is responsible for determining the posted speed on all state roadways, which should be based on engineering and traffic investigations conducted after a project is completed. The following factors should be considered in the evaluation:

1. road surface characteristics, shoulder condition, grade, alignment, and sight distance
2. 85th percentile speed and pace speed
3. roadside development and cultural/roadside friction
4. safe speed for curves within the zone
5. parking practices and pedestrian activity
6. reported crash experience for a recent 12-month period
7. design speed used for project design
8. functional classification and type of area

The selection of a posted speed limit is based on all of these factors, although the 85th-percentile speed typically serves as a basis for the maximum speed limit. The Traffic Engineering Division does not have the authority to conduct traffic investigations on non-state roadways, which are the responsibility of the local government.

2-3.0 ROADWAY CAPACITY

Roadway capacity and geometric design are interrelated. Therefore, the designer should be familiar with the basic terminology in roadway capacity and aware of the interrelationship between the two. The following subsections discuss these factors.

2-3.01 Definitions

1. <u>Capacity</u> – The maximum number of vehicles that can reasonably be expected to traverse a point or uniform section of a roadway during a given time period under prevailing roadway, traffic, and control conditions. The time period most often used for analysis is 15 minutes.

2. <u>Level of Service (LOS)</u> – A qualitative concept that has been developed to characterize acceptable degrees of congestion. In the *Highway Capacity Manual* (HCM), the qualitative descriptions of each LOS (A to E) have been converted into quantitative measures for the capacity analysis for each roadway element (e.g., mainline roadway, signalized intersection). The tables of geometric design criteria in this manual present the LOS threshold for each functional classification, which apply to all roadway elements (i.e., mainline, intersections, weaving areas).

3. <u>Average Annual Daily Traffic (AADT)</u> – The total yearly volume in both directions of travel divided by the number of days in the year.

4. <u>Average Daily Traffic (ADT)</u> – The calculation of average traffic volumes in both directions of travel in a time period greater than one day and less than one year, which is then divided by the number of days in that time period.

5. <u>Hourly Volume</u> – The total number of vehicles that pass over a given point or section of a lane or roadway during an hour.

6. <u>Peak Rate of Flow</u> – The highest equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval less than one hour, usually 15 minutes.

7. <u>Peak-Hour Factor (PHF)</u> – A ratio of the total hourly volume to the maximum 15-minute rate of flow within the hour.

8. <u>Design Hourly Volume (DHV)</u> – The one-hour volume in both directions of travel in the design year selected for determining the roadway design. Section 2-3.02 provides guidance for selecting the DHV for roadway design.

9. <u>Directional Design Hourly Volume (DDHV)</u> – The one-hour volume in one direction of travel in the selected design year.

10. <u>Design Service Volume or Flow Rate</u> – The maximum hourly vehicular volume that can pass through a roadway element at the selected LOS. The basic intent of a roadway capacity analysis is to ensure that the DHV does not exceed the calculated design service volume of the roadway element when considering the prevailing roadway, traffic, and traffic control conditions.

11. <u>Density</u> – The number of vehicles occupying a given length of lane or roadway averaged over time. Density is usually expressed as vehicles per mile (vpm).

12. Delay – The additional travel time experienced by a driver, passenger, bicyclist, or pedestrian beyond that required to travel at the desired speed. Delay is a critical performance measure on interrupted flow facilities, especially at signalized intersections. For this element, average stopped-time delay is measured, which is expressed in seconds per vehicle.

13. Speed – A rate of motion expressed as distance per unit of time, typically in miles per hour (mph).

14. Level of Service Score – A numerical output from a traveler's perception that typically indicates the average rating travelers would give a transportation facility or service under a given set of conditions.

15. Percent Time Spent Following – The average percent of total travel time that vehicles travel in platoons behind slower vehicles because of inability to pass on a 2-lane roadway.

16. Directional Distribution (D) – The division, by percent, of traffic in each direction of travel during the design hour.

17. Traffic Composition (T) – A factor that reflects the percentage of heavy vehicles (trucks, buses, and recreational vehicles) in the traffic stream during the DHV. The poorer operating capabilities and larger size of heavy vehicles should be reflected in the capacity analysis.

2-3.02 Selection of Design Year and DHV

A roadway should be designed to accommodate the traffic volume that might occur within the life of the facility under reasonable maintenance, which involves projecting the traffic conditions for a selected future year. Traffic volume projections are usually made for the 20- to 30-year range for new construction/reconstruction projects, with 20 years from the expected construction completion date being the typical selection. If multiple construction projects, the design year should be the expected completion of the final project.

Traffic volume projections for projects on existing roadways using 3R criteria are usually made based on a design year of 10 years from the expected construction completion date.

The Planning Division maintains current records of traffic data for all roadways and develops projections of expected future traffic volumes including the DHV, AADT, directional distribution, and traffic composition.

2-3.03 Impacts on Geometric Design

The following briefly discusses the interrelationship between roadway capacity and geometric design. These factors are discussed in detail in the *Highway Capacity Manual*:

1. Number of Lanes – The DHV combined with the selected LOS should determine the number of lanes on the roadway facility. The decision on the number of lanes is normally made during the planning/environmental process.

2. Traffic Composition – As the percentage of heavy vehicles increases, the roadway capacity decreases. The standard procedure is to convert the volume of trucks and recreational vehicles to a passenger car equivalent.

3. Lane and Shoulder Width – As lane and shoulder widths decrease, the roadway capacity decreases. Narrow lanes force drivers closer to vehicles in opposing lanes. Restricted shoulders force drivers to shy away from roadside obstructions. Adjustment factors should be used to calculate the influence of narrow pavement widths.

4. Alignment – Horizontal and vertical alignment may affect roadway capacity. The frequency and sharpness of curves, as well as steepness of grades, can be significant factors. For example, the capacity of a roadway is reduced where there is limited opportunity for overtaking and passing slow-moving vehicles. Adjustment factors based on the average travel speed are used to calculate the influence of grades.

5. Auxiliary Lanes – Auxiliary lanes include truck climbing lanes, turn lanes at intersections, weaving lanes at interchanges, and Continuous Two-Way Left-Turn Lanes (CTWLTL). The presence or absence of auxiliary lanes may affect the capacity of the roadway. Universal adjustment factors are not applicable and individual analyses are necessary to determine their impact on capacity.

6. At-Grade Intersections – At-grade intersections, including driveways, may have a major impact on the capacity of the roadway. In many cases, their influence on service volumes is so great that they govern the capacity of the entire roadway segment. Therefore, intersections should be evaluated individually.

7. Urban Area Development – Roadside development and adjacent land use along an urban arterial may reduce the capacity of the roadway. The factors associated with development that influence traffic flow include parking, driveways, and pedestrians.

8. Freeway Interchanges – Weaving sections and ramp terminals at interchanges may impact the capacity of the freeway mainline. By definition, these are the only allowable points of access onto a freeway. The design details of interchanges to provide sufficient capacity should be considered separately. See Chapter 7, "Grade Separations and Interchanges", for more information.

2-3.04 Capacity Analyses

The roadway mainline, intersection, or interchange should be designed to accommodate the DHV at the selected LOS, which may involve adjusting the various roadway factors that affect capacity until a design is found that will accommodate the DHV. The detailed calculations, factors, and methodologies are presented in the *Highway Capacity Manual*. The Traffic Engineering Division and/or the Planning Division should be consulted when capacity analyses are recommended.

During the analysis, the design service volume of the facility should be calculated. Capacity assumes a Level of Service E. Design service volume is the maximum volume of traffic that a projected roadway of designed dimensions is able to serve without the degree of congestion falling below a preselected level. The design should be based on the LOS as specified in the appropriate geometric design criteria table of this manual.

2-4.0 CONTROL OF ACCESS

Access control is defined as the condition where the public authority fully or partially controls the right of abutting owners to have access to and from the public roadway. Section 11-1.05 and the *Access Management Manual* present the definitions for the basic types of access control and their application.

2-5.0 CONTEXT-SENSITIVE SOLUTIONS

The Department is committed to the principles of Context-Sensitive Solutions (CSS) in order to accomplish the mission of providing a safe intermodal transportation network that is planned, designed, constructed, and maintained in an effective, cost-efficient, and environmentally sensitive manner. The Department is responsible for working with the citizens of Mississippi to create transportation facilities that accommodate all modes of transportation. A CSS approach balances safety, mobility, and the preservation of scenic, aesthetic, historic, environmental, and other community values. A CSS approach also provides insight to help each project blend into and enhance the environment by involving a full range of stakeholders throughout the planning, environmental, design, construction, and maintenance phases of the project.

2-6.0 PEDESTRIAN AND BICYCLE CONSIDERATIONS

Most transportation facilities are used by pedestrians and bicyclists to some extent; therefore, the need for sidewalk and bicycle facilities must be investigated early in project development (e.g., during the environmental process). Input from public and local officials should be gathered and considered in the decision-making process. The location of schools, churches, parks, and commercial space can influence to what extent pedestrians and bicyclists should be accommodated.

The following factors should be considered where pedestrian facilities are provided:

1. <u>Disabled Users</u> – The *PROWAG*, *MUTCD*, and this manual provide specific requirements for pedestrian facilities. The extent to which the requirements are met may vary based on the scope of the project as described below. For more information on the various scopes of work for projects, see Section 2-7.0.

    a. New Construction/Reconstruction Projects – All pedestrian facilities shall be accessible to people with disabilities in accordance with the requirements of the *PROWAG*, *MUTCD*, and this manual. The most stringent criteria should govern. See Section 14-2.06.05 for more information.

    b. Projects on Existing Roadways – Projects on existing roadways that are categorized as 3R projects shall be accessible to people with disabilities to the maximum extent feasible. Exceptions may be granted due to environmental constraints or undue burdens, such as significant right of way and/or utility impacts. In these cases, the designer should make every effort to provide reasonable access to disabled pedestrians. See Section 12-3.07 for more information.

    c. Preventive Maintenance Projects – Projects on existing roadways that are intended to extend the serviceable life of the roadway (i.e., pavement surface) and enhance roadway safety may also require upgrading of pedestrian facilities to be accessible to disabled pedestrians. See Section 12-6.0 for more information.

For deviations from the requirements for pedestrian facilities, see Section 2-10.0.

2. Capacity – Above-minimum values for sidewalk widths, paved shoulder widths, and other roadway features should be considered where moderate to heavy pedestrian and bicycle volumes can be expected.

3. Bridges – Pedestrian and bicycle accommodations should be considered, except where they are prohibited, when determining shoulder widths or sidewalks on bridges.

4. Construction/Maintenance – If existing sidewalks are disturbed during construction or maintenance activities, alternative pedestrian routes shall be provided during this work.

2-7.0 PROJECT SCOPE OF WORK

2-7.01 Scope of Work Determination

The scope of work for a proposed roadway project is a key control in design, and it begins with the Location Committee Meeting. A general description of the committee and its procedures is provided in S.O.P. No. ENV-03-01-00-000. The Location Committee is responsible for recommending the project scope of work, which in turn determines the type of environmental action needed. Listed below are the three classes of environmental action, along with a brief description of each:

- Class I – Environmental Impact Study (EIS) – A project is expected to have a significant environmental impact. The resulting document from this study will be the Record of Decision (ROD).

- Class II – Categorical Exclusion (CE) – A project is not expected to have a significant environmental impact.

- Class III – Environmental Assessment (EA) – The significance of the environmental impact of a project is not clearly established. This study should lead to one of two possible results. If significant impacts are discovered, then this study is elevated to an EIS. Otherwise, the resulting document from the EA should be the FONSI (Finding of No Significant Impact).

For more information on classes of environmental action, S.O.P. No. ENV-02-01-01-160 should be referenced.

A Location Committee Report should be prepared by the District after the Location Committee meeting to document the discussions that resulted in the recommended scope of work for the project. All feasible alternatives for accomplishing the goals of the project should be included in the report, and justification should be provided for choosing the recommended alternative as well as for eliminating other alternatives.

The RWD-600 form should also be included in the report. This form includes multiple types of information about the proposed project, including the scope of the project, the functional classification of the roadway, design traffic data, and the geometric design criteria to be used for the design of the project.

The following subsections contain descriptions that provide general definitions for the project scope of work.

2-7.02 New Construction/Reconstruction

The Department's geometric design criteria for new construction/reconstruction are presented in the geometric design criteria tables in Section 2-9.0 (rural roadways) and Section 14-2.0 (urban roadways). New construction consists of constructing a roadway on new location. Reconstruction of an existing roadway typically includes the addition of travel lanes and/or reconstruction of the existing horizontal and vertical alignment, but essentially within the existing roadway corridor. The Department uses the same set of criteria for new construction and reconstruction projects.

Any at-grade intersections that fall within the limits of a new construction or reconstruction project are also considered new construction or reconstruction. Chapter 6, "At-Grade Intersections", presents the Department's design criteria for the new construction/reconstruction of at-grade intersections. Typical improvements at existing intersections may include, but are not limited to:

1. adding through and auxiliary lanes for approaches
2. redesigning the turning radii to accommodate the design vehicle
3. flattening the approach and intersection gradients
4. realigning the angle of intersection
5. channelizing the intersection
6. converting the intersection to a roundabout
7. converting the intersection to a Restricted Crossing U-turn intersection (J-turn)

2-7.03 3R

Chapter 12, "Existing Roadways", presents the Department's 3R criteria (resurfacing, restoration, and rehabilitation) for improvements to existing non-freeway and freeway facilities. The use of 3R criteria is primarily intended to extend the service life of the existing roadway, enhance roadway safety, and make cost-efficient improvements to the existing geometrics. 3R criteria typically allow for less acquisition of right of way than new construction/reconstruction criteria. 3R improvements for non-freeway facilities may include, but are not limited to:

1. pavement resurfacing or rehabilitation (including shoulders)
2. lane and shoulder widening
3. placement of rumble strips/stripes
4. flattening horizontal or vertical curves
5. widening the roadside clear zone
6. flattening side slopes
7. improving intersection geometry or alignment
8. converting an existing CTWLTL to a raised median
9. adding a CTWLTL
10. retrofitting an existing roadway to add other modes of travel
11. converting a non-curbed urban street into a curbed street
12. revising the location, spacing, or design of existing driveways along the mainline

13. adding or removing parking spaces
14. adding sidewalks, curb cuts, or accessible pedestrian signals
15. replacing existing pedestrian facilities to meet *ADA* criteria
16. railroad crossing improvements
17. upgrading guardrail and other safety hardware to meet current criteria
18. upgrading signs
19. geometric and/or safety improvements to existing bridges
20. drainage improvements

3R improvements for freeway facilities may include:

1. pavement resurfacing or rehabilitation (including shoulders)
2. placement of rumble strips/stripes
3. widening the roadside clear zone
4. flattening side slopes
5. improvements to interchange gore areas
6. upgrading guardrail and other safety hardware to meet current criteria
7. upgrading signs
8. geometric (e.g., vertical clearance, shoulder width) and/or safety improvements to existing bridges within the project limits
9. drainage improvements
10. addition of auxiliary lanes
11. lengthening existing acceleration or deceleration lanes
12. realigning or widening an existing ramp
13. fence replacement

Where extensive reconstruction is deemed necessary for freeways (e.g., realignment, significant right of way acquisition, adding through lanes), new construction criteria should be used.

2-7.04 Preventive Maintenance

Chapter 12, "Existing Roadways", presents the Department's criteria for resurfacing (1R) projects on non-freeway facilities. Further information can also be found in the "MDOT Pavement Preservation & Preventive Maintenance Treatment Policy for Federal-Aid Projects".

2-7.05 Spot Improvements (Non-Freeways)

Chapter 12, "Existing Roadways", presents the Department's criteria for the geometric design of spot improvement projects.

2-7.06 Other Projects

Other project scopes of work may include new installations or replacement of existing roadway lighting, traffic signals, Intelligent Transportation Systems (ITS), signing, etc.

2-8.0 OVERALL BASIS FOR DESIGN CRITERIA

Geometric design criteria, including controlling design criteria as presented in Section 2-9.02, set forth throughout this manual are typically expected to be met in project designs. Procedures for deviations from these criteria are discussed in Section 2-10.0.

2-8.01 Functional Classification

The functional classification of the roadway is the basic control for the Department's geometric design criteria. The classification system, discussed in Section 2-1.0, establishes the general type and character of the roadway. For example, a freeway indicates paved shoulders, control of access, relatively high design speeds, and wide clear zones. Collector roads usually have non-paved shoulders, access control by regulation, and lower design speeds.

2-8.02 Alignment Elements

The geometric design criteria for the following alignment elements on rural roadways are presented in Section 2-9.0. Below is additional information on each of the alignment elements:

1. Horizontal Curvature and Superelevation – For a given design speed, the maximum superelevation rate and side-friction factor determine the minimum allowable radius. Chapter 3, "Horizontal Alignment", presents the design details for horizontal curvature and superelevation. See the *Standard Drawings* for more information.

2. Vertical Curvature – Vertical curvature design involves both crest and sag vertical curves. Crest vertical curves should be designed such that the line of sight is not obstructed by the crest of the vertical curves. Sag vertical curves should be designed such that the headlight sight distance is equal to or greater than the stopping sight distance. Chapter 4, "Vertical Alignment", discusses the length of vertical curves based on sight distance and design speed. Chapter 6, "At-Grade Intersections", and Chapter 7, "Grade Separations and Interchanges", discuss sight distance for at-grade intersections and interchanges, respectively.

3. Vertical Clearance – Vertical clearance is measured between the bottom of overhead structures and the finished roadway or railway surface. The minimum clearance for roadways applies across the entire roadway width, including auxiliary lanes. The minimum clearance for railroads typically applies across the entire railroad right of way.

4. Stopping Sight Distance (SSD) – Sight distance is the length of roadway ahead that is visible to the driver. The minimum distance available should be sufficient to enable a vehicle traveling at the design speed to stop before reaching an object in its path. SSD is the sum of two distances — the brake reaction distance and the braking distance. The AASHTO publication *A Policy on Geometric Design of Highways and Streets* discusses these concepts and procedures in detail.

   Sight distance may be restricted by vertical alignment, horizontal alignment, roadside obstructions, or any combination of these elements. Minimum SSD values are provided in the geometric design criteria tables in Section 2-9.0, Chapter 12, and Chapter 14. However, an attempt should be made to provide greater sight distance than the minimum. The use of SSD in the design of horizontal and vertical curves is discussed in Chapter 3, "Horizontal Alignment", and Chapter 4, "Vertical Alignment." SSD is measured from the driver's eye height 3.5 feet above the pavement to a 2.0-foot object height.

5. Decision Sight Distance (DSD) – Drivers must sometimes make decisions where information is difficult to perceive or where unexpected maneuvers are required. Roadway elements, traffic volumes, and traffic control devices may compete for the driver's attention in areas of concentrated demand. Example locations include interchange ramp exits, complex intersections, and changes in cross section (e.g., lane drops). In these cases, DSD may be warranted to allow the driver more time to evaluate the roadway condition and select a course of action. Table 4-5-B provides values for DSD. The distance is measured from a 3.5-foot eye height to a 2.0-foot object height. Chapter 7, "Grade Separations and Interchanges", provides additional information on DSD at exit and entrance ramps.

6. Passing Sight Distance (PSD) – PSD considerations are limited to 2-lane, 2-way roadways on which vehicles frequently overtake slower moving vehicles and the passing must be accomplished on a lane used by opposing traffic. PSD values provided in Table 4-5-C are based on the distance needed to safely complete a normal passing maneuver. Where feasible, PSD should be provided over a high proportion of the roadway length. Additional lanes or adjustments to the horizontal and vertical alignment may be warranted in restricted areas. PSD is measured from a 3.5-foot eye height to a 3.5-foot object height. Section 10-1.01 provides the Department's pavement marking criteria for no-passing zones.

7. Intersection Sight Distance (ISD) – At each intersection, the potential exists for vehicles to conflict with one another when entering, exiting, or crossing the intersection. Sufficient sight distance should be provided for a driver to perceive these potential conflicts and to perform the necessary actions needed to negotiate the intersection safely. Section 6-6.0 provides information on ISD.

8. Grades – Maximum grades for each design speed are based on the functional classification of the roadway and the terrain conditions.

2-8.03 Cross-Section Elements

The geometric design criteria for the following cross-section elements on rural roadways are presented in Section 2-9.0. Figure 2-8-A illustrates a typical roadway cross section. Below is additional information on each of the cross-section elements:

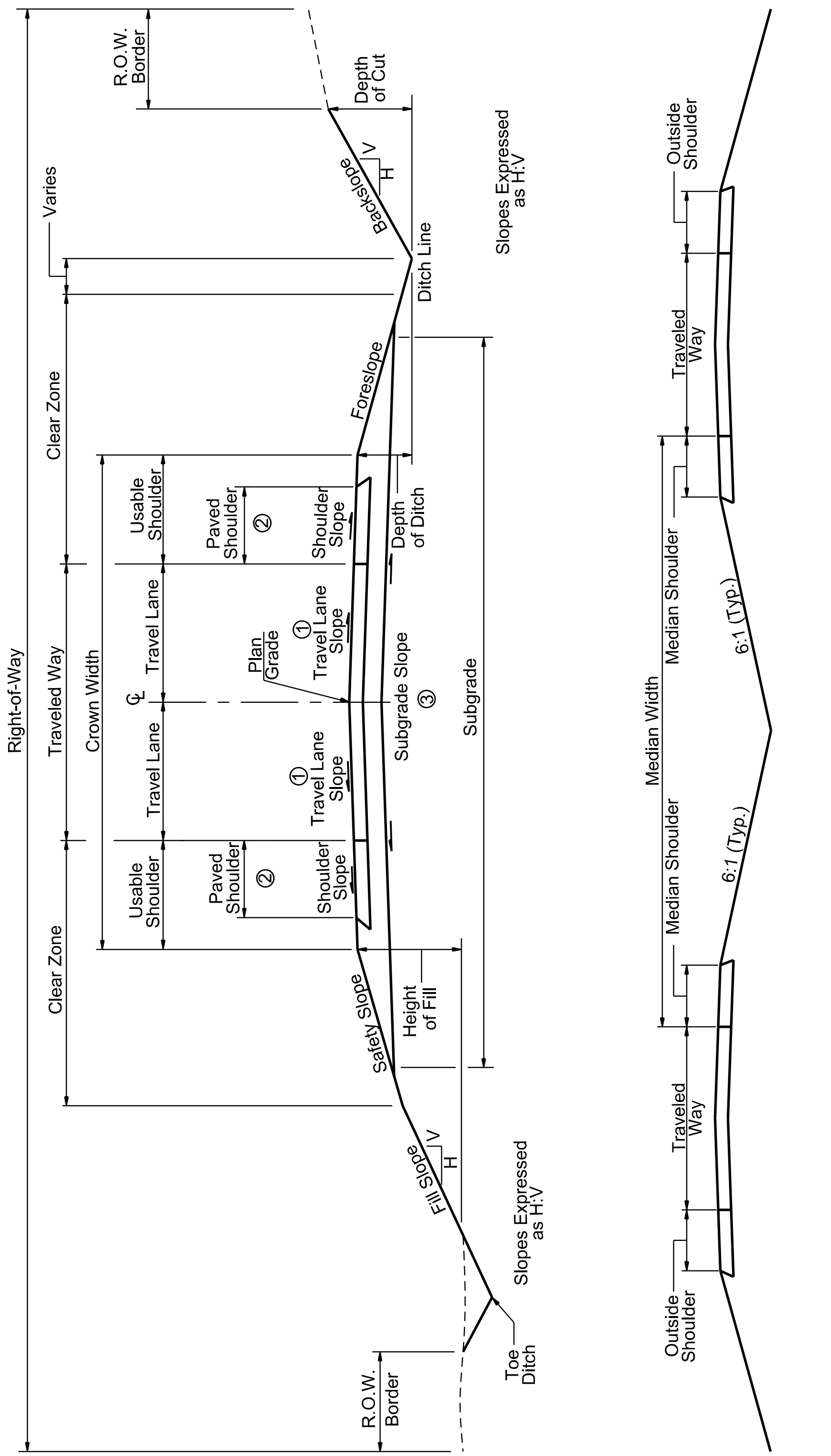
1. Traveled Way – The portion of the roadway for the movement of vehicles is called the traveled way. Shoulders, curbs, turn lanes, and bicycle lanes are not included in the traveled way.

2. Lane Width – The lane width of a roadway influences operational characteristics and driver comfort. The use of 12-foot lanes is predominant on high-speed roadways ($V \geq 50$ miles per hour) but should also be the goal on low-speed roadways ($V \leq 45$ miles per hour) unless right of way constraints prohibit this width.

3. Auxiliary Lane Width – Auxiliary lanes are the portion of the roadway adjoining the traveled way intended for purposes supplementary to the through traffic movement. Auxiliary lanes include truck climbing lanes and turning lanes at intersections. Auxiliary lanes should be as wide as adjacent through lanes.

4. Usable Shoulder Width – The shoulder is that portion of the cross section contiguous to the traveled way that is available for stopped vehicles, emergencies, and lateral support of the pavement structure. Shoulders also provide a recovery area for vehicles that inadvertently leave the traveled way. The usable shoulder width is measured from the edge of the traveled way to the intersection of the shoulder slope and the foreslope (or safety slope) planes.

5. Paved Shoulder Width – Paved shoulders provide a better all-weather riding surface and load support than afforded by native soils or stabilized materials. Therefore, they are provided on all freeway facilities and considered on other multilane arterials. A minimum 2-foot paved shoulder should be provided on all non-curbed facilities.



Notes:
Typical cross slopes:
① *Concrete and Bituminous: 2%*
② *Paved Shoulders ≤ 4 feet: 2%*
Paved Shoulders > 4 feet: 4%
Non-paved Shoulders: 4%
③ *Subgrade: 2%*

For urban roadway typical sections, see Figure 14-2-C and Figure 14-2-D.

TYPICAL SECTION NOMENCLATURE
Figure 2-8-A

6. Median Width – The median on a divided roadway separates opposing traffic, reduces headlight glare, and assists in the safe operation of crossing vehicles at intersections and crossovers. The median width is measured between the inside edges of the two traveled ways of the opposing roadways. On rural multilane roadways, the median width is typically 64 feet. On rural freeways, the median width is also typically 64 feet; however, on urban freeways with right of way restrictions, a 64-foot median width may not be feasible. As a minimum, the median on urban freeways shall be wide enough to accommodate the minimum left shoulder plus the space needed for a concrete median barrier. Wider medians should be considered to accommodate future widening.

7. Right of Way Width – The right of way width is the sum of all cross-section elements, including traveled way width, shoulder widths, median width, side slopes, clear zones, ditches, border widths, and frontage roads where provided. The right of way border width is defined as the distance between the right of way lines inward to the toe of fill slopes or top of cut slopes.

8. Bridge Width – Bridge width refers to the clear width measured between the gutter lines or bridge rails, whichever is less. The geometric design criteria tables in Section 2-9.0, Chapter 12, "Existing Roadways", and Chapter 14, "Geometric Design of Urban Roadways", provide the Department's bridge width criteria for new/reconstructed bridges and existing bridges to remain in place.

9. Roadway Width at Bridge Ends – Protective devices are typically installed at approach bridge ends within the clear zone. In some cases, widening the embankment at bridge ends to facilitate installation of protective devices may be necessary. For the typical transition between the normal roadway width and the roadway width at bridge ends, the designer should refer to the *Standard Drawings*.

10. Roadside Clear Zone – An adequate clearance should be provided between the edge of traveled way and roadside obstructions (e.g., trees, retaining walls, drainage structures, fences) to allow drivers of out-of-control vehicles a reasonable distance to recover and return to the roadway. Breakaway luminaire supports and sign posts are permitted within the roadside clear zone. See Chapter 9, "Roadside Safety", for more information.

11. Travel Lane and Shoulder Cross Slope – The pavement surface should be sloped sufficiently to ensure proper drainage but without affecting vehicular operation. For roadways with three or more lanes sloped in the same direction, the slope of an outer travel lane may be increased to 2.5% for all lanes beyond the first two lanes. However, the outer lane(s) should not be less than that of the adjacent travel lane. The shoulder cross slope should not be less than the cross slope of the adjacent traveled way.

12. Side Slopes – Side slopes provide enhanced roadway stability and a reasonable opportunity for drivers of out-of-control vehicles to regain control. The rate of slope depends on the functional classification, safety considerations, and maintenance, but may also be influenced by high-volume-change (HVC) soil (if present) and terrain.

    In cut sections, the definitions of cross-section slopes are:

a. Foreslope – The slope extending outward and downward from the shoulder to the ditch line.

b. Backslope – The slope extending upward and outward from the ditch line to intersect the natural ground line.

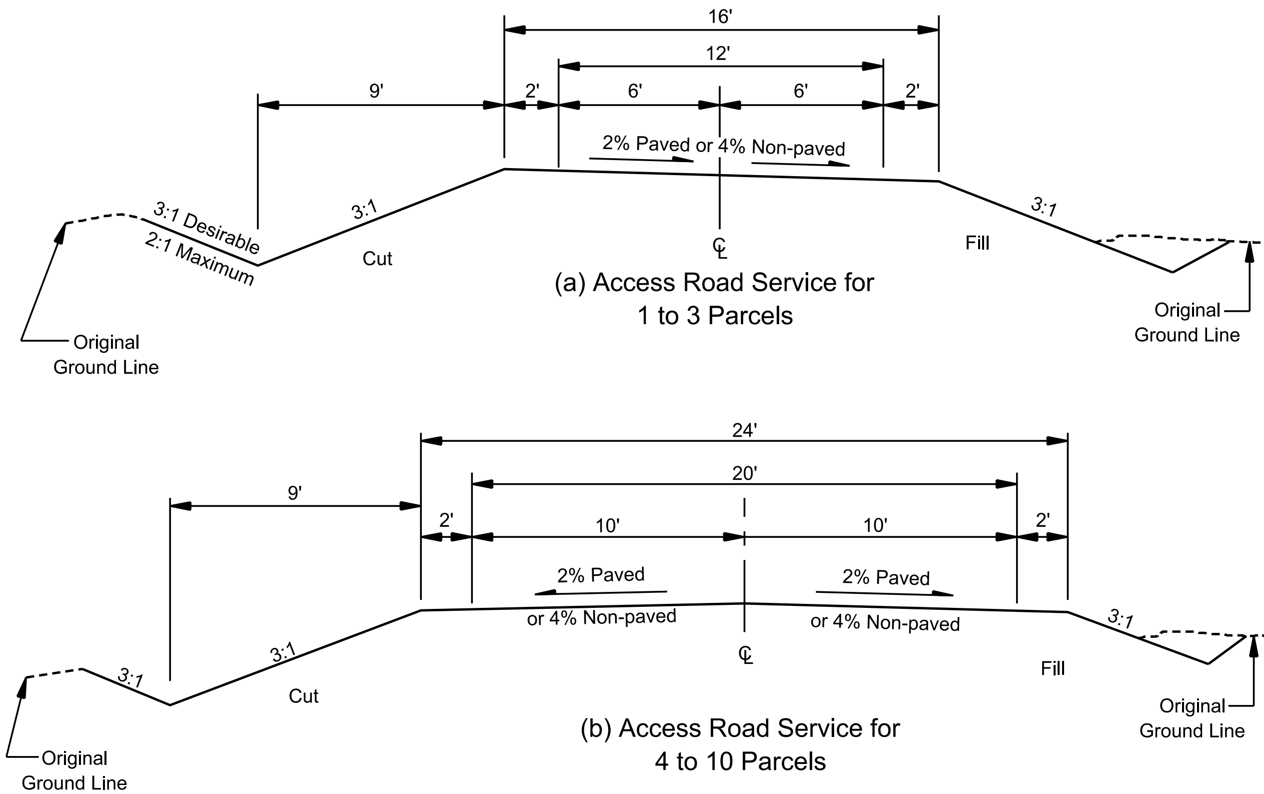
In fill sections, the definitions of cross-section slopes are:

a. Safety Slope – The slope extending outward and downward from the shoulder point to a horizontal distance equal to the clear zone distance (see Section 9-2.0).

b. Fill Slope – The embankment slope extending outward and downward from the outer limit of the safety slope to intersect the natural ground line.

The cross section at the ditch bottoms and tops of backslopes should be rounded. Typically, toe ditches at the bottom of fill slopes should be provided.

13. Access/Frontage Roads – Figure 2-8-B illustrates typical sections for access roads servicing one to 10 parcels. For access roads serving more than 10 parcels, the designer should use the cross section shown in Figure 2-8-B(b), but with a crown width of 30 feet and with 12-foot travel lanes. Existing access roads should be replaced in kind or in accordance with Figure 2-8-B, whichever is greater.



ACCESS/FRONTAGE ROAD TYPICAL SECTION NOMENCLATURE
Figure 2-8-B

2-9.0 GEOMETRIC DESIGN CRITERIA

2-9.01 Design Criteria Elements

Geometric design criteria for the various types of construction projects (new construction/reconstruction/3R) are presented throughout this manual. Specific design criteria for design controls, cross-section elements, and alignment elements are presented in the following geometric design criteria tables in this section, Chapter 12, “Existing Roadways”, and Chapter 14, “Geometric Design of Urban Roadways”:

New Construction/Reconstruction (Rural)
1. Table 2-9-A (freeways)
2. Table 2-9-B and Table 2-9-C (arterials)
3. Table 2-9-D and Table 2-9-E (collectors)
4. Table 2-9-F (local roads)

3R Construction (Rural and Urban)
1. Table 12-2-C and Table 12-2-D (rural arterials)
2. Table 12-2-E and Table 12-2-F (rural collectors)
3. Table 12-2-G (rural local roads)
4. Table 12-3-A and Table 12-3-B (urban arterials)
5. Table 12-3-C and Table 12-3-D (urban collectors)
6. Table 14-2-J (urban local roads)
7. Table 12-4-A (rural freeways)
8. Table 12-4-B (urban freeways)

New Construction/Reconstruction (Urban)
1. Table 14-2-E (freeways)
2. Table 14-2-F and Table 14-2-G (arterials)
3. Table 14-2-H and Table 14-2-I (collectors)
4. Table 14-2-J (local roads)

Each of these tables is followed by a set of footnotes that provides additional information for the items listed in the criteria tables.

The criteria provided in Table 2-9-A, as well as the additional information provided in the footnotes for this table, meet or exceed the criteria in the AASHTO publication *A Policy on Design Standards – Interstate System*. The design for all rural interstates shall be in compliance with these criteria.

2-9.02 Controlling Design Criteria

While all design criteria contained in this manual are important, they do not all equally affect the safety and operations of a roadway. Therefore, the following 10 criteria have been identified as controlling design criteria elements for the design of projects:

1. design speed
2. lane width
3. shoulder width
4. horizontal curve radius

5. superelevation rate
6. maximum grade
7. SSD (e.g., horizontal alignment, K-values for crest vertical curves)
8. cross slope for travel lanes and shoulders
9. vertical clearance
10. design loading structural capacity

Application of the ten controlling design criteria is as follows:

- NHS Routes with design speeds $\geq$ 50 miles per hour – All 10 controlling design criteria apply. Deviations from the design criteria values for any of these ten controlling design criteria are to be evaluated and documented in accordance with the procedures for Design Exceptions set forth within Section 2-10.0.

- NHS Routes with design speeds $\leq$ 45 miles per hour – Only design speed and design loading structural capacity apply. Deviations from the design criteria for either of these two design criteria are to be evaluated and documented in accordance with the procedures for Design Exceptions set forth within Section 2-10.0. Deviations from the design criteria for the remaining eight design criteria are to be evaluated and documented in accordance with the procedures for Design Variances set forth within Section 2-10.0.

- Non-NHS Routes – Deviations from any of these 10 controlling design criteria are to be evaluated and documented in accordance with the procedures for Design Variances set forth within Section 2-10.0.

See Section 2-10.0 for more information on design deviations.

2-9.03 Non-Controlling Design Criteria

Any design criteria that are not considered controlling design criteria are considered to be non-controlling criteria. See Section 2-10.0 for more information on design deviations.

Table 2-9-A
GEOMETRIC DESIGN CRITERIA FOR RURAL FREEWAYS
(TWO LANES EACH DIRECTION)
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	Rural
Design Controls	Design Year			2-3.02	20 years
	*Design Speed			2-2.01	70 mph
	Control of Access			11-1.05	Full (Type 1)
	Level of Service Threshold			2-3.04	C
Cross-Section Elements	*Travel Lane Width			2-8.03	12 ft
	Outside Shoulder Width **(1)**		*Usable	2-8.03	12 ft
			*Paved		10 ft
	Median Shoulder Width **(1)**		*Usable	2-8.03	8 ft
			*Paved		4 ft
	*Cross Slope		Travel Lane **(2)**	2-8.03	2%
			Shoulder		See Note **(3)**
	Auxiliary Lanes		Lane Width	2-8.03	12 ft
			Shoulder Width		Paved: 10 ft Usable 12 ft
	Median Width		Depressed	2-8.03	64 ft Minimum
			Concrete Median Barrier		N/A
	New and Reconstructed Bridges		*Design Loading Structural Capacity	2-8.03	HL-93
			Minimum Width **(4)**		Traveled Way + 12 ft (outside shoulder) + 6 ft (median shoulder)
	Existing Bridges to Remain in Place **(5)**		*Design Loading Structural Capacity	2-8.03 12-2.03	See Note **(5a)**
			Minimum Width **(5b)**		Traveled Way + 10 ft (outside shoulder) + 4 ft (median shoulder)
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			2-8.03 11-1.01	30 ft
	Roadside Clear Zone		Guardrail	2-8.03 9-2.0	Usable Shoulder Width
			Obstruction **(6)**		30 ft
	Slope Schedule **(7)**	Cut	Foreslope (within clear zone)	2-8.03	6:1
			Depth of Ditch		4 ft
			Backslope		3:1
		Fill	Safety Slope (within clear zone)		6:1
			Fill Slope (outside clear zone)		3:1
Alignment Elements	**DESIGN SPEED**				**70 mph**
	*Stopping Sight Distance			2-8.02	730 ft
	Decision Sight Distance **(8)**			2-8.02 4-5.02	1105 ft
	*Superelevation Rate			3-4.01	e_{max} = 10%
	*Minimum Horizontal Curve Radius			3-3.0	1630 ft
	*Maximum Grades	Level		2-8.02 4-3.01	3%
		Rolling			4%
	Minimum Grades			4-3.02	See Note **(9)**
	Vertical Curve (K-values)		*Crest	2-8.02 4-5.0	247
			Sag		181
	*Vertical Clearance (freeway under) **(10)**		New/Reconstructed Bridges	2-8.02 2-10.03	Desirable: 17 ft Minimum: 16 ft
			Existing Bridges		Desirable: 17 ft Minimum: 16 ft
			Sign Truss/ Pedestrian Bridge		19 ft
	Vertical Clearance (freeway over railroad) **(11)**			2-8.02	25 ft

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Table 2-9-A

1. Shoulder Width –

a. The minimum paved shoulder widths shown in Table 2-9-A are consistent with the AASHTO publication A Policy on Design Standards – Interstate System*. Paved shoulder widths on interstates shall not be less than these widths. The table below includes these widths, but also provides additional information regarding usable shoulder widths, freeways with three or more lanes in one direction, and truck traffic.*

Number of Lanes in One Direction	*Left*		*Right*	
	Usable	*Paved*	*Usable*	*Paved*
2	*8 ft*	*4 ft*	*12 ft**	*10 ft**
3 or more	*12 ft**	*10 ft**	*12 ft**	*10 ft**

**Where truck traffic exceeds 250 DDHV, consideration should be given to increasing these widths by two feet.*

b. Paved shoulders on non-interstate freeways wider than the typical 2-foot width should not be provided except as approved for special conditions.

c. A minimum paved shoulder width of 10 feet, desirably 12 feet, should be provided for a distance of 350 feet beyond a reduction of the basic number of lanes (lane drop) for a recovery area for drivers. See Section 7-2.05 for more information.

2. Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0% and shall not be less than 1.5%. For freeways with three or more lanes sloped in the same direction, the cross slope may be increased to 2.5% for all lanes beyond the first two lanes that are sloped in the same direction. The cross slope of an outer lane shall not be less than that of the adjacent lane.

3. Cross Slope of Shoulder (normal crown sections) – The cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%. The portion of the shoulder that is not paved should have a cross slope of 4%. The cross slope of the shoulder shall not be less than that of the adjacent travel lane.

4. New and Reconstructed Bridge Width –

a. The width shown in the table applies to a 1-way bridge of a divided freeway. The width should be increased for auxiliary lanes and for wider shoulders as shown in Footnote 1 above.

b. For bridges less than or equal to 200 feet in length, the bridge width shall be no less than the full paved width of the approach roadway when the minimum width provided in the table cannot be met.

5. *Existing Bridges to Remain in Place –*

   a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

   b. *The width provided in the table applies to a 1-way existing bridge that is less than or equal to 200 feet in length. For existing bridges that are greater than 200 feet in length, the minimum shoulder width shall be no less than four feet for both the left and the right sides.*

   c. *For existing bridges that are in compliance with the applicable widths as outlined in the table or in footnote 5b, consideration should be given to widening the bridges to comply with the new construction width, especially in areas with high traffic volumes.*

6. *Roadside Clear Zone – The width shown in the table is for design speeds of 60 miles per hour to 70 miles per hour for 6:1 fill slopes or cut foreslopes and for ADTs greater than 6000. Section 9-2.0 provides clear zone distances for other roadside conditions. All values are measured from the edge of the traveled way, except where long (greater than 0.5 miles) auxiliary lanes are present.*

7. *Slopes – If high-volume-change (HVC) soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

8. *Decision Sight Distance (DSD) – The distance shown in the table is for a speed/path/direction change on a rural freeway.*

9. *Minimum Grades –*

   a. *On roadways, level grades are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

   b. *On bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

10. *Vertical Clearance (freeway under) –*

    a. *The vertical clearance shown in the table shall be provided over the entire freeway width, including shoulders, interchange ramps, and collector-distributor (C-D) roadways.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the freeway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

11. *Vertical Clearance (freeway over railroad) – The vertical clearance shown in the table should typically be provided over the entire railroad right of way width.*

Table 2-9-B
GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS (2-LANE)
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	Design DHV Under 200	Design DHV 201 – 400	Design DHV Over 400
Design Controls	Design Year			2-3.02	20 years		
	*Design Speed			2-2.01	65 mph		
	Control of Access			11-1.05	Control by Regulation (Type 3)		
	Level of Service Threshold			2-3.04	C		
Cross-Section Elements	*Travel Lane Width			2-8.03	12 ft		
	Shoulder Width		*Usable	2-8.03	6 ft	8 ft	10 ft
			Paved **(1)**		2 ft		
	*Cross Slope		Travel Lane **(3)**	2-8.03	2%		
			Shoulder		See Note **(4)**		
	Auxiliary Lanes		Lane Width	2-8.03	12 ft		
			Usable Shoulder Width		6 ft	8 ft	10 ft
	New and Reconstructed Bridges		*Design Loading Structural Capacity	2-8.03	HL-93		
			Minimum Width **(6)**		36 ft	40 ft	44 ft
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	2-8.03 12-2.03	See Note **(7)**		
			Minimum Width				
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			2-8.03 11-1.01	15 ft – 20 ft		
	Roadside Clear Zone		Guardrail	2-8.03 9-2.0	Usable Shoulder Width		
			Obstruction		See Note **(8)**		
	Slope Schedule **(9)**	Cut	Foreslope (within clear zone)	2-8.03	6:1		
			Depth of Ditch		3 ft		
			Backslope		3:1		
		Fill	Safety Slope (within clear zone)		6:1		
			Fill Slope (outside clear zone)		3:1		
Alignment Elements	**DESIGN SPEED**				**65 mph**		
	*Stopping Sight Distance			2-8.02	645 ft		
	Intersection Sight Distance **(10)**			6-6.0	720 ft		
	*Superelevation Rate			2-8.02 3-4.01	e_{max} = 10%		
	*Minimum Horizontal Curve Radius			2-8.02 3-3.0	1340 ft		
	*Maximum Grades		Level	2-8.02 4-3.01	3%		
			Rolling		4%		
	Minimum Grades			4-3.02	See Note **(11)**		
	Vertical Curve (K-values)		*Crest	2-8.02 4-5.0	193		
			Sag		157		
	*Vertical Clearance (arterial under) **(12)**		New/Reconstructed Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft		
			Existing Bridges		Desirable: 17 ft Minimum: 16 ft		
			Sign Truss/ Pedestrian Bridge		19 ft		
	Vertical Clearance (arterial over railroad) **(13)**			2-8.02	25 ft		

*For application of controlling design criteria, see Section 2-9.02.

Table 2-9-C
GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS (MULTILANE)
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	Multilane
Design Controls	Design Year			2-3.02	20 years
	*Design Speed			2-2.01	65 mph
	Control of Access			11-1.05	Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)
	Level of Service Threshold			2-3.04	C
Cross-Section Elements	*Travel Lane Width			2-8.03	12 ft
	Outside Shoulder Width		*Usable	2-8.03	10 ft
			Paved **(1)**		2 ft
	Median Shoulder Width		*Usable **(2)**	2-8.03	8 ft
			Paved **(1)**		2 ft
	*Cross Slope		Travel Lane **(3)**	2-8.03	2%
			Shoulder		See Note **(4)**
	Auxiliary Lanes		Lane Width	2-8.03	12 ft
			Usable Shoulder Width		Same Usable/Paved Width Adjacent to Travel Lane
	Median Width		Depressed **(5)**	2-8.03	64 ft Typical
	New and Reconstructed Bridges		*Design Loading Structural Capacity	2-8.03	HL-93
			Minimum Width **(6)**		Traveled Way + 10 ft (outside shoulder) + 6 ft (median shoulder)
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	2-8.03 12-2.03	See Note **(7)**
			Minimum Width		
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			2-8.03 11-1.01	30 ft
	Roadside Clear Zone		Guardrail	2-8.03 9-2.0	Usable Shoulder Width
			Obstruction		See Note **(8)**
	Slope Schedule **(9)**	Cut	Foreslope (within clear zone)	2-8.03	6:1
			Depth of Ditch		4 ft
			Backslope		3:1
		Fill	Safety Slope (within clear zone)		6:1
			Fill Slope (outside clear zone)		3:1
Alignment Elements	**DESIGN SPEED**				**65 mph**
	*Stopping Sight Distance			2-8.02	645 ft
	Intersection Sight Distance			6.6.0	See Note **(10)**
	*Superelevation Rate			2-8.02 3-4.01	e_{max} = 10%
	*Minimum Horizontal Curve Radius			2-8.02 3-3.0	1,340 ft
	*Maximum Grades		Level	2-8.02 4-3.01	3%
			Rolling		4%
	Minimum Grades			4-3.02	See Note **(11)**
	Vertical Curve (K-values)		*Crest	2-8.02 4-5.0	193
			Sag		157
	*Vertical Clearance (arterial under) **(12)**		New/Reconstructed Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft
			Existing Bridges		Desirable: 17 ft Minimum: 16 ft
			Sign Truss/ Pedestrian Bridge		19 ft
	Vertical Clearance (arterial over railroad) **(13)**			2-8.02	25 ft

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 2-9-B and 2-9-C

1. *Shoulder Surface Type (outside and median shoulders) –*

   a. *Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved for special conditions.*

   b. *A minimum paved shoulder width of 10 feet, desirably 12 feet, should be provided for a distance of 350 feet beyond a reduction of the basic number of lanes (lane drop) of a multilane arterial for a recovery area for drivers. See Section 7-2.05 for more information.*

2. *Median Shoulder Width – For three or more lanes in each direction, consideration should be given to increasing the usable median shoulder width to 10 feet.*

3. *Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0%. For arterials with three or more lanes sloped in the same direction, the cross slope may be increased to 2.5% for all lanes beyond the first two lanes that are sloped in the same direction. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

4. *Cross Slope of Shoulder (normal crown sections) –*

   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

5. *Median Width – The 64-foot typical median width applies to new construction or to the dualization of an existing 2-lane, 2-way roadway. Exceptions to this width may be considered for the following:*

   a. *areas with environmental restrictions*

   b. *roadways with Type 1 access control (See Chapter 11, "Right of Way and Fencing".)*

   c. *roadways where full-width bridges have already been constructed at a lesser width*

   d. *other areas with justification*

6. *New and Reconstructed Bridge Width –*

   a. *The widths shown in Table 2-9-B apply to a 2-lane arterial. The widths for the lower traffic volumes may be increased up to 44 feet if large equipment (i.e., farm equipment) is expected to cross the bridge. These widths should also be increased for auxiliary lanes.*

   b. *The width shown in Table 2-9-C applies to a 1-way bridge of a divided arterial. The width should be increased for auxiliary lanes. For undivided arterials, the width should equal the approach roadway width.*

c. *For bridges less than or equal to 200 feet in length, the width should be no less than the full paved width of the approach roadway when the minimum widths provided in the tables cannot be met.*

7. *Existing Bridges to Remain in Place –*

   a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

   b. *Undivided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side.*

   c. *Divided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side for each roadway.*

   d. *The minimum widths provided in Footnotes 7b and 7c do not include auxiliary lanes or other existing features.*

   e. *For existing bridges that are in compliance with the applicable widths in footnotes 7b, 7c, and 7d, consideration should be given to widening the bridges to comply with the new construction width, especially in areas with high traffic volumes.*

8. *Roadside Clear Zone – The recommended clear zones are based on design speed, side slopes, and traffic volumes. Section 9-2.0 provides additional information. All values are measured from the edge of the traveled way, except where long (greater than 0.5 miles) auxiliary lanes are present.*

9. *Slopes – If high-volume-change (HVC) soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

10. *Intersection Sight Distance (ISD) – The value provided in Table 2-9-B assumes ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways.*

11. *Minimum Grades –*

    a. *On roadways, level grades are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    b. *On bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

12. *Vertical Clearance (arterial under) –*

    a. *The vertical clearance shown in the table should be provided over the entire arterial roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the arterial roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

d. Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).

13. <u>Vertical Clearance (arterial over railroad</u>) – The vertical clearance shown in the table should typically be provided over the entire railroad right of way width.

Table 2-9-D
GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (2-LANE)
(New Construction/Reconstruction)

	DESIGN ELEMENT		Manual Section	Design ADT: Under 400	Design ADT: 400 – 1,500	Design ADT: 1,500 – 2,000	Design ADT: Over 2,000
Design Controls	Design Year		2-3.02	20 Years			
Design Controls	*Design Speed **(1)**		2-2.01	55 mph			Level: 60 mph Rolling: 55 mph
Design Controls	Control of Access		11-1.05	Control by Regulation (Type 3)			
Design Controls	Level of Service Threshold		2-3.04	C			
Cross-Section Elements	*Travel Lane Width		2-8.03	12 ft			
Cross-Section Elements	Shoulder Width	*Usable	2-8.03	4 ft	5 ft	6 ft	8 ft
Cross-Section Elements	Shoulder Width	Paved **(2)**	2-8.03	2 ft			
Cross-Section Elements	*Cross Slope	Travel Lane **(3)**	2-8.03	2%			
Cross-Section Elements	*Cross Slope	Shoulder	2-8.03	See Note **(4)**			
Cross-Section Elements	Auxiliary Lanes	Lane Width	2-8.03	11 ft			
Cross-Section Elements	Auxiliary Lanes	Usable Shoulder Width	2-8.03	4 ft	5 ft	6 ft	8 ft
Cross-Section Elements	New and Reconstructed Bridges	*Design Loading Structural Capacity	2-8.03	HL-93			
Cross-Section Elements	New and Reconstructed Bridges	Minimum Width **(6)**	2-8.03	32 ft	34 ft	36 ft	40 ft
Cross-Section Elements	Existing Bridges to Remain in Place	*Design Loading Structural Capacity	2-8.03 12-2.03	See Note **(7)**			
Cross-Section Elements	Existing Bridges to Remain in Place	Minimum Width	2-8.03 12-2.03	See Note **(7)**			
Cross-Section Elements	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)		2-8.03 11-1.01	15 ft – 20 ft			
Cross-Section Elements	Roadside Clear Zone	Guardrail	2-8.03 9-2.0	Usable Shoulder Width			
Cross-Section Elements	Roadside Clear Zone	Obstruction	2-8.03 9-2.0	See Note **(8)**			
Cross-Section Elements	Slope Schedule **(9)** – Cut	Foreslope (within clear zone)	2-8.03	V ≤ 55 mph 4:1 V > 55 mph 6:1			
Cross-Section Elements	Slope Schedule **(9)** – Cut	Depth of Ditch	2-8.03	3 ft			
Cross-Section Elements	Slope Schedule **(9)** – Cut	Backslope	2-8.03	3:1			
Cross-Section Elements	Slope Schedule **(9)** – Fill	Safety Slope (within clear zone)	2-8.03	4:1			
Cross-Section Elements	Slope Schedule **(9)** – Fill	Fill Slope (outside clear zone)	2-8.03	3:1			

	DESIGN SPEED		Manual Section	55 mph	60 mph
Alignment Elements	*Stopping Sight Distance		2-8.02	495 ft	570 ft
Alignment Elements	Intersection Sight Distance **(10)**		6-6.0	610 ft	665 ft
Alignment Elements	*Superelevation Rate		2-8.02 3-4.01	e_{max} = 10%	
Alignment Elements	*Minimum Horizontal Curve Radius		2-8.02 3-3.0	877 ft	1090 ft
Alignment Elements	*Maximum Grades	Level	2-8.02 4-3.01	6%	5%
Alignment Elements	*Maximum Grades	Rolling	2-8.02 4-3.01	7%	6%
Alignment Elements	Minimum Grades		4-3.02	See Note **(11)**	
Alignment Elements	Vertical Curve (K-values)	*Crest	2-8.02 4-5.0	114	151
Alignment Elements	Vertical Curve (K-values)	Sag	2-8.02 4-5.0	115	136
Alignment Elements	* Vertical Clearance (collector under) **(12)**	New/Reconstructed Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft	
Alignment Elements	* Vertical Clearance (collector under) **(12)**	Existing Bridges	2-8.02	Desirable: 16 ft Minimum: 14.5 ft	
Alignment Elements	* Vertical Clearance (collector under) **(12)**	Sign Truss/ Pedestrian Bridge	2-8.02	19 ft	
Alignment Elements	Vertical Clearance (collector over railroad) **(13)**		2-8.02	25 ft	

*For application of controlling design criteria, see Section 2-9.02.

Table 2-9-E
GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (MULTILANE)
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	Multilane
Design Controls	Design Year			2-3.02	20 years
	*Design Speed **(1)**			2-2.01	65 mph
	Control of Access			11-1.05	Control by Regulation (Type 3)
	Level of Service Threshold			2-3.04	C
Cross-Section Elements	*Travel Lane Width			2-8.03	12 ft
	Outside Shoulder Width		*Usable	2-8.03	10 ft
			Paved **(2)**		2 ft
	Median Shoulder Width		*Usable	2-8.03	8 ft
			Paved **(2)**		2 ft
	*Cross Slope		Travel Lane **(3)**	2-8.03	2%
			Shoulder		See Note **(4)**
	Auxiliary Lanes		Lane Width	2-8.03	12 ft
			Usable Shoulder Width		Same Usable/Paved Width Adjacent to Travel Lane
	Median Width		Depressed **(5)**	2-8.03	64 ft Typical
	New and Reconstructed Bridges		*Design Loading Structural Capacity	2-8.03	HL-93
			Minimum Width **(6)**		Traveled Way + 10 ft (outside shoulder) + 6 ft (median shoulder)
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	2-8.03 12-2.03	See Note **(7)**
			Minimum Width		
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			2-8.03 11-1.01	30 ft
	Roadside Clear Zone		Guardrail	2-8.03 9-2.0	Usable Shoulder Width
			Obstruction		See Note **(8)**
	Slope Schedule **(9)**	Cut	Foreslope (within clear zone)	2-8.03	6:1
			Depth of Ditch		4 ft
			Backslope		3:1
		Fill	Safety Slope (within clear zone)		6:1
			Fill Slope (outside clear zone)		3:1
Alignment Elements	**DESIGN SPEED**				**65 mph**
	*Stopping Sight Distance			2-8.02	645 ft
	Intersection Sight Distance			6-6.0	See Note **(10)**
	*Superelevation Rate			2-8.02 3-4.01	e_{max} = 10%
	*Minimum Horizontal Curve Radius			2-8.02 3-3.0	1340 ft
	*Maximum Grades		Level	2-8.02 4-3.01	4.5%
			Rolling		5.5%
	Minimum Grades **(11)**			4-3.02	Desirable: 0.4% Minimum: 0.0%
	Vertical Curve (K-values)		*Crest	2-8.02 4-5.0	193
			Sag		157
	*Vertical Clearance (collector under) **(12)**		New/Reconstructed Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft
			Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft
			Sign Truss/ Pedestrian Bridge		19 ft
	Vertical Clearance (collector over railroad) **(13)**			2-8.02	25 ft

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 2-9-D and 2-9-E

1. *Design Speed – A design speed as low as the posted speed limit may be used.*

2. *Shoulder Surface Type (outside and median shoulders) – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved for special conditions.*

3. *Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0%. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

4. *Cross Slope of Shoulder (normal crown sections) –*

   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

5. *Median Width – The 64-foot typical median width applies to new construction or to the dualization of an existing 2-lane, 2-way roadway. Exceptions to this width may be considered for the following:*

   a. *areas with environmental restrictions*

   b. *roadways with Type 1 access control (See Chapter 11, "Right of Way and Fencing".)*

   c. *roadways where full-width bridges have already been constructed at a lesser width*

   d. *other areas with justification*

6. *New and Reconstructed Bridge Width –*

   a. *The widths shown in Table 2-9-D apply to a 2-lane collector. These widths may be increased up to 44 feet if large equipment (i.e., farm equipment) is expected to cross the bridge. These widths should also be increased for auxiliary lanes.*

   b. *The width shown in Table 2-9-E applies to a 1-way bridge of a divided collector. The width should be increased for auxiliary lanes. For undivided roadways, the width should equal the approach roadway width.*

   c. *For bridges less than or equal to 200 feet in length, the bridge width should be no less than the full paved width of the approach roadway when the minimum widths provided in the tables cannot be met.*

   d. *For bridges greater than 200 feet in length, a minimum shoulder width of four feet may be used on both the left and the right sides.*

7. *Existing Bridges to Remain in Place:*

   a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

   b. *Undivided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side.*

   c. *Divided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side for each roadway.*

   d. *Minimum widths provided in Footnotes 7b and 7c do not include auxiliary lanes or other existing features.*

   e. *For existing bridges that are in compliance with the applicable widths in footnotes 7b, 7c, and 7d, consideration should be given to widening the bridges to comply with the new construction width, especially in areas with high traffic volumes.*

8. *Roadside Clear Zone – The recommended clear zones are based on design speed, side slopes, and traffic volumes. Section 9-2.0 provides additional information. All values are measured from the edge of the traveled way, except where long (greater than 0.5 miles) auxiliary lanes are present.*

9. *Slopes – If high-volume-change soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

10. *Intersection Sight Distance (ISD) – The values provided in Table 2-9-D assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways.*

11. *Minimum Grades –*

    a. *On roadways, level grades are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    b. *On bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

12. *Vertical Clearance (collector under) –*

    a. *The vertical clearance shown in the table should be provided over the entire collector roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the collector roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

13. *<u>Vertical Clearance (collector over railroad)</u> – The vertical clearance shown in the table should typically be provided over the entire railroad right of way width.*

Table 2-9-F
GEOMETRIC DESIGN CRITERIA FOR RURAL LOCAL ROADWAYS
(New Construction/Reconstruction)

	DESIGN ELEMENT		Manual Section	Design ADT				
				Under 400		400 – 2,000		Over 2,000
Design Controls	*Design Speed **(1)**		2-2.01	30 mph		30 – 45 mph		See Table 2-9-D
	Control of Access		11-1.05	Control by Regulation (Type 3)				
	Level of Service Threshold		2-3.04	C				
Cross-Section Elements	*Travel Lane Width		2-8.03	10 ft		11 ft		
	Shoulder Width	*Usable	2-8.03	4 ft		6 ft		
		Paved **(2)**		2 ft		2 ft		
	*Cross Slope	Travel Lane **(3)**	2-8.03	2%				
		Shoulder		See Note **(4)**				
	Auxiliary Lanes	Lane Width	2-8.03	10 ft		11 ft		
		Usable Shoulder Width		4 ft		6 ft		
	New and Reconstructed Bridges	*Design Loading Structural Capacity	2-8.03	HL-93				
		Minimum Width **(5)**		26 ft or Approach Roadway Width (whichever is greater)				
	Existing Bridges to Remain in Place	*Design Loading Structural Capacity	2-8.03 12-2.03	See Note **(6)**				
		Minimum Width						
	Roadside Clear Zone	Guardrail	2-8.03 9-2.0	Usable Shoulder Width				
		Obstruction		See Note **(7)**				
	Slope Schedule **(8)**	Cut Foreslope	2-8.03	3:1		4:1		
		Depth of Ditch		3 ft				
		Backslope		3:1				
		Fill Slope		Desirable: 4:1 Minimum: 3:1				
Alignment Elements	**DESIGN SPEED**			**30 mph**	**35 mph**	**40 mph**	**45 mph**	
	*Stopping Sight Distance		2-8.02	200 ft	250 ft	305 ft	360 ft	
	Intersection Sight Distance **(9)**		6-6.0	335 ft	390 ft	445 ft	500 ft	
	*Superelevation Rate		2-8.02 3-4.01	e_{max} = 6%				
	*Minimum Horizontal Curve Radius (e_{max} = 6%)		2-8.02 3-3.0	231 ft	340 ft	485 ft	643 ft	
	*Maximum Grades	Level	2-8.02 4-3.01	7%				
		Rolling		8%				
	Minimum Grades		4-3.02	See Note **(10)**				
	Vertical Curve (K-values)	*Crest	2-8.02 4-5.0	19	29	44	61	
		Sag		37	49	64	79	
	*Vertical Clearance (local roadway under) **(11)**	New/Reconstructed Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft				
		Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft				
	Vertical Clearance (local roadway over railroad) **(12)**		2-8.02	25 ft				

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Table 2-9-F

1. *Design Speed – A design speed as low as the posted speed limit may be used.*

2. *Shoulder Surface Type – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved for special conditions.*

3. *Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0%. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

4. *Cross Slope of Shoulder (normal crown sections) –*

    a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

    b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

    c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

    d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

5. *New and Reconstructed Bridge Width – The width provided in Table 2-9-F applies to a 2-lane local roadway. This width should be increased for additional lanes.*

6. *Existing Bridges to Remain in Place –*

    a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications.

    b. *The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side.*

    c. *The minimum width provided in Footnote 6b does not include auxiliary lanes or other existing features.*

    d. *The existing bridge width may remain unchanged if there is no crash pattern associated with the existing width. However, consideration should still be given to widening or reconstructing the bridge. If a crash pattern exists, the bridge should be widened or reconstructed to 26 feet or the approach roadway width, whichever is greater.*

7. *Roadside Clear Zone – The recommended clear zones are based on design speed, side slopes, and traffic volumes. Section 9-2.0 provides additional information. All values are measured from the edge of the traveled way, except where auxiliary lanes are present.*

8. *Slopes – If high-volume-change (HVC) soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

9. *<u>Intersection Sight Distance (ISD)</u> – The values provided in Table 2-9-F assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways.*

10. *<u>Minimum Grades</u> –*

    a. *On roadways, level grades are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    b. *On bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

11. *<u>Vertical Clearance (local roadway under)</u> –*

    a. *The vertical clearance shown in the table should be provided over the entire local roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the local roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

12. *<u>Vertical Clearance (local roadway over railroad)</u> – The vertical clearance shown in the table should typically be provided over the entire railroad right of way width.*

2-10.0 EXCEPTIONS/VARIANCES TO DESIGN CRITERIA

2-10.01 Design Deviation Definitions

A Design Exception Request is defined as a request for approval of a proposed design value for any of the controlling design criteria that are not compliant with the values provided in the geometric design criteria tables.

A Design Variance Request is defined as a request for approval of a proposed non-compliant design value for any design elements that are not identified as controlling design criteria. Design Variance requests include, but are not limited to, requests for deviations from the following:

- any advisory conditions presented in this manual that are not included in the geometric design criteria tables in this chapter or in Chapters 12 and 14
- any recommendations provided in other publications including, but not limited to, those listed in Section 1-4.0

2-10.02 Design Exception and Design Variance Applications

The following information explains the various conditions when Design Exceptions and Design Variances may or may not be necessary. These conditions are based on the scope of the project and if the geometric design criteria element in question is a controlling design criteria element. See Section 2-9.02 for information on the application of controlling design criteria elements.

A Design Exception or Design Variance Request may not be necessary in the following situations:

- When transitioning from the approved geometric design criteria of a proposed project to the existing geometric elements of the roadway outside of the project limits, non-compliant criteria may be necessary to achieve this transition within a reasonable distance. Extending the project limits to make further improvements to the roadway is not necessary for existing geometric elements that are performing satisfactorily. 3R criteria is preferable for such transitions, but sound engineering judgment should be used to provide the safest, most aesthetically pleasing, but feasible, transition possible. This exemption applies to projects both on the NHS and those not on the NHS. See Chapter 12, “Existing Roadways”, for more information.

- Each project has a specific purpose and need. The scope of work for some projects may be to improve or add certain elements to an existing roadway without improving every geometric element within the project limits. Any elements that are excluded from the project scope but that are performing satisfactorily may be exempt from requiring a Design Variance request. However, this exemption applies only to projects that are not on the NHS.

A Design Exception is necessary if a controlling design criteria element is not included in the scope of the project, or if the proposed design value for the controlling design criteria element is non-compliant (excluding transition areas).

Non-controlling design criteria elements that are included in the project scope but that have non-compliant proposed design values will be at the discretion of the Roadway Design Division

Engineer. A conceptual design should be submitted to the Roadway Design Division Engineer to obtain advanced approval prior to the designer proceeding beyond conceptual design. The Roadway Design Division Engineer will determine if Chief Engineer approval in the form of a Design Variance is necessary. If Chief Engineer approval is deemed not necessary, detailed documentation, including approval by the Roadway Design Division Engineer, should still be included in the project file. Such documentation should include, but is not limited to, the following information:

1. project description
2. route and functional classification
3. urban or rural
4. work classification (i.e., new construction/reconstruction, 3R, etc.)
5. proposed non-compliant value
6. justification for not meeting the limiting criteria

If the non-controlling design criteria element for which the proposed non-compliant value is being requested is not a roadway design element, the Roadway Design Division Engineer should forward the request to the appropriate Division Engineer for approval.

2-10.03 Department Procedures

After determining that a Design Exception or Design Variance is necessary, a request should be prepared that identifies the recommended design value and the proposed deviation from that value. Justification for the deviation should be documented in the project file.

A standard form, "Design Exception/Design Variance Request", is located on the Department's website, and this form identifies the following information that should be included in a Design Exception or a Design Variance request:

1. The impacts of meeting the design criteria should be investigated. Factors to evaluate and to document using the standard request form include, but are not limited to:

    a. Design Exception

        i. Purpose of project
        ii. Project schedule (i.e., current letting date)
        iii. Existing conditions (e.g., roadway section, posted speed, design traffic data)
        iv. Specific design criteria that will not be met
        v. Alternatives considered
        vi. Justification for not meeting the limiting criteria, such as, but not limited to:

            1) scope of the project
            2) proposed design value
            3) constraints
            4) compatibility with adjacent sections of roadway

        vii. Comparison of the safety and operational performance of the roadway and other impacts, such as:

1) right of way
2) community
3) environmental
4) cost
5) usability by all modes of transportation

viii. Probable timing for any future improvements within (or adjacent to) the project limits

ix. Proposed mitigation measures (e.g., additional signing, pavement marking)

NOTE: Any item that is compliant with other design resources but does not meet MDOT criteria should be included.

b. Design Exception for Vertical Clearance on the Interstate

i. An additional process is required when any construction projects result in a vertical clearance less than 16 feet at any bridges over the interstate. The Department should coordinate directly with the Surface Deployment and Distribution Command Transportation Agency (SDDCTEA) in such cases. A standard form, "Interstate Vertical Clearance Exception Coordination", is located on the Department's website, and this form identifies the information that should be included in this coordination and the contact information for the SDDCTEA.

ii. These coordination efforts will be in addition to the normal Design Exception approval process that takes place within MDOT for any non-compliant controlling criteria on projects on the NHS.

c. Design Variance

i. Purpose of project
ii. Project schedule (i.e., current letting date)
iii. Existing conditions (e.g., roadway section, posted speed, design traffic data)
iv. Specific design criteria that will not be met
v. Alternatives considered
vi. Justification for not meeting the limiting criteria, such as, but not limited to:

1) scope of the project
2) proposed design value
3) constraints
4) compatibility with adjacent sections of roadway

vii. Probable timing for any future improvements within or adjacent to the project limits

d. Design Variance for *ADA* Criteria

i. Purpose of project

ii. Project schedule (current letting date)
iii. Existing conditions of pedestrian facilities
iv. Specific design criteria that will not be met
v. Justification for not meeting the limiting criteria, such as, but not limited to:

1) scope of the project
2) proposed design value
3) constraints
4) compatibility with adjacent sections of roadway

vi. Alternative design solutions considered to provide access within the pedestrian circulation path, justification for eliminating these alternatives, documentation of coordination efforts with local activists, and supporting documentation from the design manual and/or the *PROWAG*

vii. Documentation that the non-compliant item has been added to the agency's transition plan and when that item is scheduled to be brought into compliance

NOTE: Any item that is compliant with the PROWAG or other design resources but does not meet Department criteria should be included in the Design Variance request.

2. All Design Exception and Design Variance requests should be submitted for approval to the Chief Engineer by the Roadway Design Division Engineer. For Federal-aid projects under FHWA full oversight, approved Design Exception requests should be forwarded to FHWA for informational purposes.

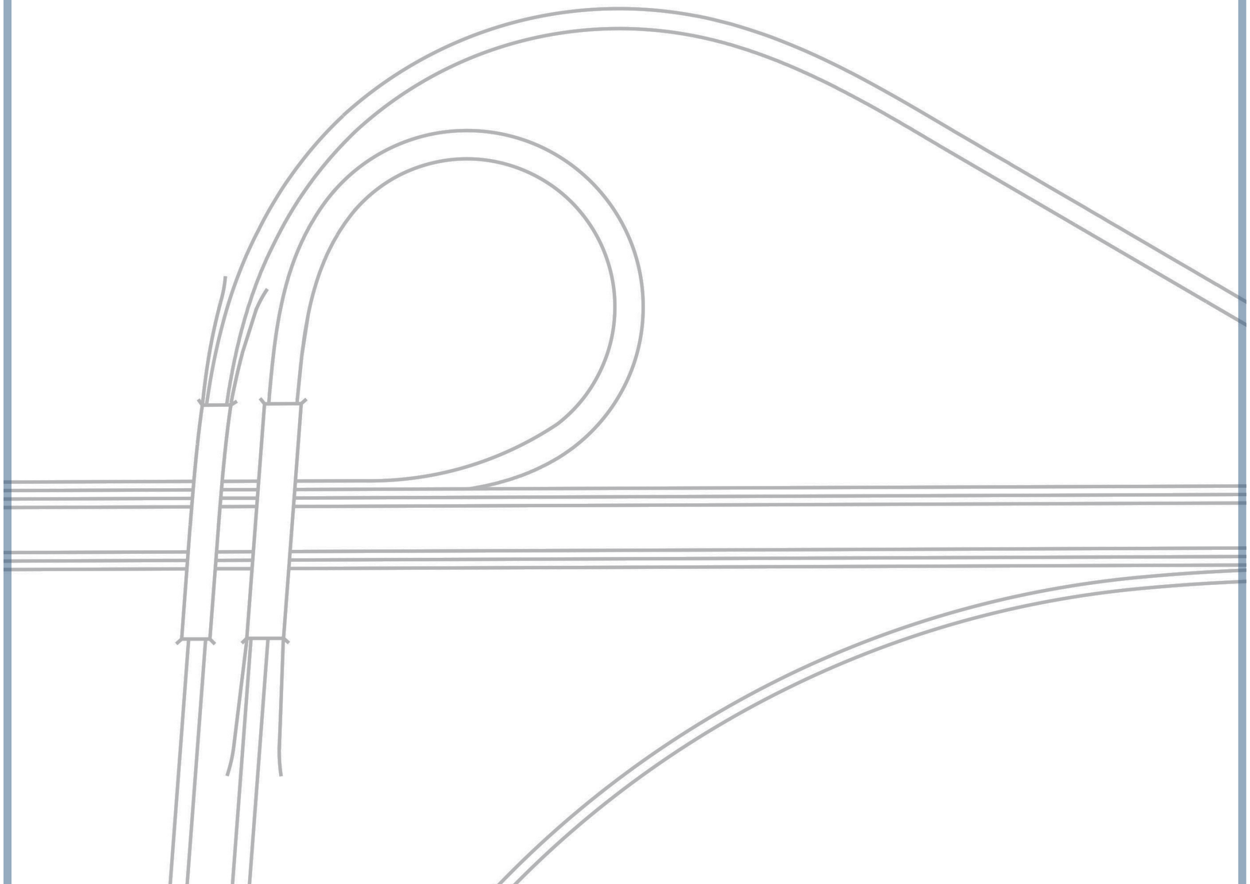
3. The documentation for the Design Exception or Design Variance should become part of the project file. Section 1-5.0 provides guidance on the project file.

2-11.0 VALUE ENGINEERING

Value engineering is a process in which the value of a project's intent is optimized by crafting a mix of performance and costs. A value engineering study identifies unnecessary expenditures and/or determines alternative cost-efficient designs, thereby increasing the value of the project. For guidance on when and how a value engineering study should be conducted, the Department's "Value Engineering Policy" should be referenced.

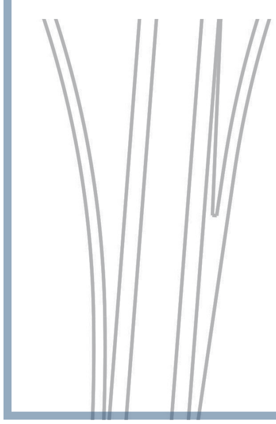
2-12.0 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2018.
2. *Highway Capacity Manual 2010*, Transportation Research Board, 2010.
3. *A Policy on Design Standards Interstate System*, AASHTO, 2016.
4. *Mitigation Strategies for Design Exceptions*, FHWA, 2007.
5. *Federal Register*, "Revision of Thirteen Controlling Criteria for Design and Documentation of Design Exceptions," May 5, 2016.



CHAPTER 3

Horizontal Alignment



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Chapter 3
HORIZONTAL ALIGNMENT

This chapter discusses the details for the proper design of permanent horizontal alignments. Much of the criteria are equally applicable to rural and urban roads. However, the horizontal alignment for low-speed urban roadways (V ≤ 45 miles per hour) involves special considerations, which are discussed in Section 14-2.04.

3-1.0 ESTABLISHING AND ADJUSTING ALIGNMENTS

The alignment established by the Location Committee or provided in the Environmental Document will normally be adhered to for projects on new location. However, circumstances encountered during the design process may require the alignment to be adjusted. In such cases, the District and every Division that was involved with setting the original alignment should be included in any discussions related to adjusting the alignment.

Projects that are on existing roadways typically require a survey of the existing alignment. Adjustments to the existing alignment for design purposes should not be made without prior approval of the District.

3-2.0 GENERAL CRITERIA

Many design criteria for horizontal alignment (e.g., design speed, minimum radii, superelevation rates) are specific and can be measured. Other factors cannot be defined and require judgment and consideration of existing conditions. General controls for horizontal alignment that should be considered, if feasible, include the following:

1. The alignment should be as directional as feasible and consistent with physical and economic constraints. A winding alignment composed of short curves should be avoided.

2. The alignment should be consistent. Sharp curves at the ends of long tangents and sudden changes from areas of flat curvature to sharp curvature should be avoided.

3. Sharp curves for small deflection angles should be avoided. Curves should be a minimum of 500 feet long for a deflection angle of five degrees or less. In constrained areas, small deflection angles of 20 minutes or less may be allowed without a horizontal curve. On high-speed (V ≥ 50 miles per hour), controlled-access facilities, a desirable length of curve should be approximately 30 times the design speed, or L = 30V, where V is expressed in miles per hour.

4. Sharp curves on long, high fills should be avoided. Under such conditions, it is difficult for drivers to perceive the extent of horizontal curvature.

5. Abrupt reversals in alignment (reverse curves) should be avoided. There should be a sufficient tangent distance between curves to ensure proper superelevation transition for each curve, as well as sufficient distance for signing.

6. Broken-back curves (short tangent between two curves in the same direction) should be avoided except where topographical or right of way constraints make other alternatives infeasible. The preferred approach is to use one longer, flatter curve.

7. Sharp horizontal curvature near crest or sag vertical curves should be avoided. Such combinations of curvature can limit sight distance. Horizontal alignment should be coordinated with the existing or proposed vertical alignment. The designer should ensure that Decision Sight Distance (DSD) is provided for drivers approaching a change in horizontal alignment whereby it is not obscured by a crest vertical curve. Examples include realignments or detours for bridge replacement sites, interchange exit ramp deceleration lanes and gore areas, and horizontal alignment improvements on existing roadways. If it is not feasible to attain DSD approaching a detour, then the designer should, at a minimum, provide Stopping Sight Distance (SSD).

8. If feasible, horizontal curves should not begin or end on a bridge, nor should there be any superelevation transition on a bridge. Where it is not feasible to avoid a horizontal curve or superelevation transition on a bridge, the Roadway Design Division should coordinate with the Bridge Design Division.

9. On new location projects, skewed bridge alignments at stream, roadway, or railway crossings should be avoided where feasible. Skews may have a significant impact on bridge design, construction, and maintenance. Preferably, the roadway centerline should be aligned so that the bridges are approximately perpendicular to the crossings. However, the skew at any type of crossing should preferably not be greater than 45 degrees.

3-3.0 HORIZONTAL CURVES

3-3.01 Curve Types and Selection

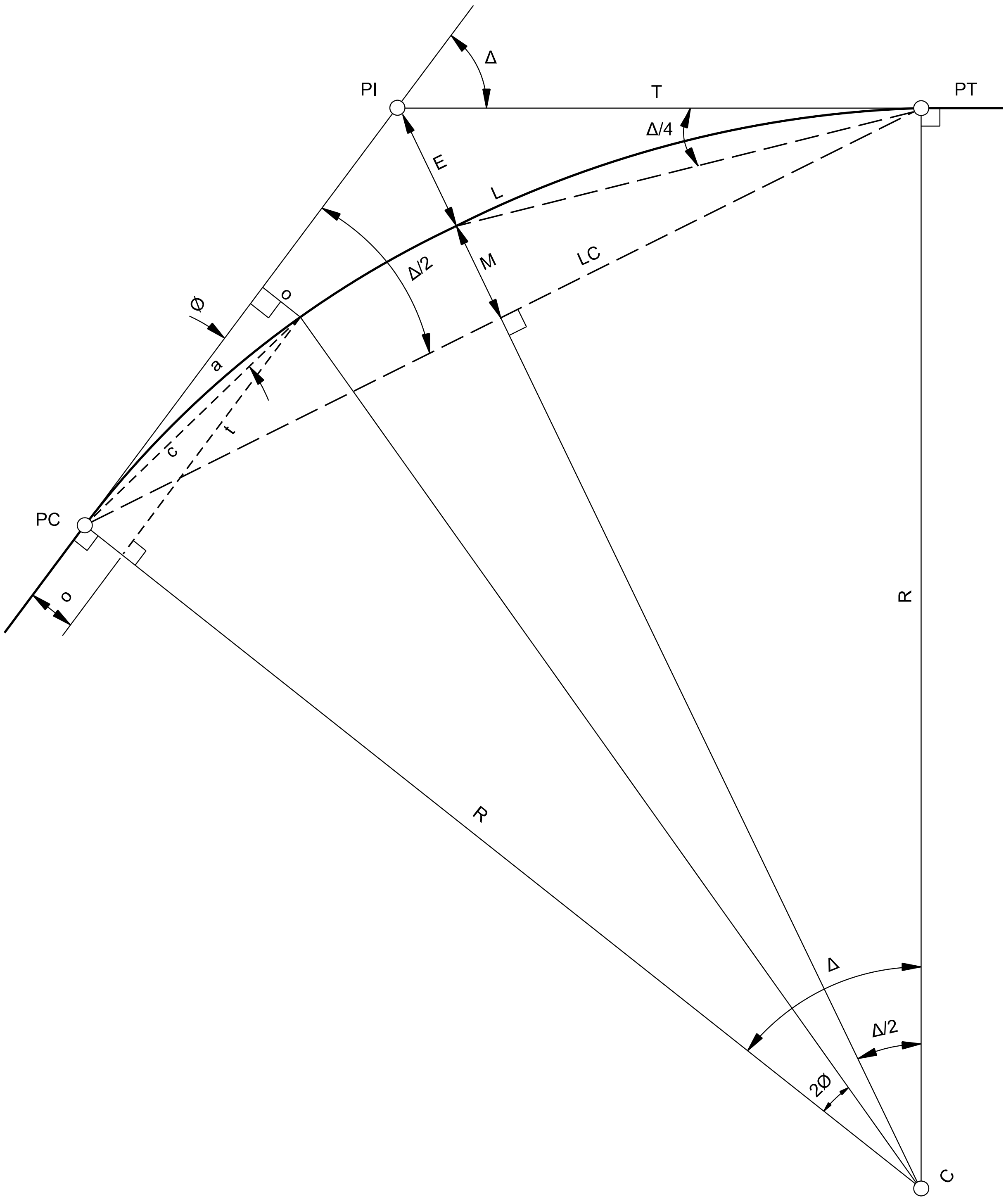
Deflection changes in alignment generally are accomplished by using a simple curve or, where necessary, a compound curve.

3-3.01.01 Simple Curves

The following information should apply to simple curves:

1. Layout – A simple curve is that portion of the arc of a circle that achieves the desired deflection without using an entering or exiting transition. Figure 3-3-A illustrates a typical simple curve layout. The minimum radius for various design speeds is discussed in Section 3-4.01.

2. Computations – The PI station, deflection angle (Δ), and minimum radius (R) are usually used for establishing simple curves. The remaining curve data can be computed using the formulas in Figure 3-3-A.



TYPICAL SIMPLE CURVE LAYOUT
Figure 3-3-A

PC = Point of Curve (beginning of curve)
PT = Point of Tangency (end of curve)
PI = Point of Intersection of Tangents
PRC = Point of Reverse Curve
PCC = Point of Compound Curve
Δ = Deflection angle (degrees) = central angle of curve
T = Tangent distance (feet) T = distance from PC to PI or distance from PI to PT
L = Length of curve (feet) L = distance from PC to PT along curve
R = Radius of curve (feet)
D = Degree of curvature (100-foot arc definition) (degrees)
E = External distance (PI to midpoint of curve) (feet)
C = Intersection of radii at center of circular arc
LC = Length of long chord (PC to PT) (feet)
M = Middle ordinate (midpoint of arc to midpoint of long chord) (feet)
a = Length of arc to any point on a curve (feet)
c = Length of chord from PC to any point on curve (feet)
Ø = Deflection angle from tangent to any point on curve (degrees)
t = Distance along tangent from PC to any point on curve (feet)
o = Tangent offset to any point on curve (feet)

$$D = 100\frac{(360/2\pi)}{R} = 5729.577961/R$$

$$T = R(\tan(\Delta/2)) = R = \frac{\sin(\Delta/2)}{\cos(\Delta/2)}$$

$$L = \frac{\Delta}{360} = 2\pi R = 100\,\Delta/D$$

$$E = \frac{R}{\cos(\Delta/2)} - R = T\tan(\Delta/4)$$

$$LC = 2R\sin(\Delta/2) = 2T\cos(\Delta/2)$$

$$M = R\left(1 - \cos(\Delta/2)\right) = E\cos(\Delta/2)$$

$$a = \frac{(200\text{Ø})(2\pi R)}{100(360)} = \frac{(\text{Ø})(\pi R)}{90}$$

$$c = 2R\left(\sin\frac{(100)(360a)}{(200)(2\pi R)}\right) = 2R\left(\sin\frac{90a}{\pi R}\right)$$

$$\text{Ø} = \frac{90a}{(\text{Ø})(\pi R)}$$

$$\cos\text{Ø} = \frac{(R - o)}{2R}$$

$$t = R\sin 2\text{Ø} = (c)\cos\text{Ø}$$

$$o = (C)\sin\text{Ø}$$

$$o = R - \sqrt{R^2 - t^2}$$

$$o = R - (R\cos 2\text{Ø})$$

$$o = R(1 - \cos 2\text{Ø})$$

$$\pi = 3.141592654$$

TYPICAL SIMPLE CURVE LAYOUT
Figure 3-3-A
(Continued)

3. Stationing – Curve stationing should be determined as follows:

   a. PC Station = PI Station – T
   b. PT Station = PC Station + L

   Figure 3-3-A defines the terms.

4. Accuracy (For Closure Purposes) – The curve data should preferably be calculated and recorded to the following level of accuracy:

   a. all linear distances, including the radius (R), to the nearest one thousandth of a foot

   b. the deflection angle (Δ) to the nearest one thousandth of a second

   c. the degree of curvature (D) to the nearest tenth of a second

3-3.01.02 Spiral Curves

Spiral curves should not be used on new construction projects. Spiral curves on existing alignments may remain in place. The following information applies to spiral curves:

1. Layout – Spiral curves provide a gradual vehicular transition into and out of circular curves without a sharp break from the tangent sections. The spiral also provides a convenient arrangement for the superelevation runoff length. Typical design practice is to use the superelevation runoff length for the length of the spiral curve (L_S). Superelevation runoff lengths are discussed in Section 3-4.02.

2. Computation – The parts of a spiral curve can be calculated with the use of a Department approved computer program, such as GEOPAK or OpenRoads.

3. Stationing – Curve stationing should be determined as follows:

   a. Master PI Station – T_s = TS Station
   b. TS Station + L_s = SC Station
   c. SC Station + L_c = CS Station
   d. CS Station + L_s = ST Station

4. Accuracy – The accuracy of curve data for simple curves also applies to spiral curves. See Section 3-3.01.01.

3-3.01.03 Compound Curves

Compound curves are sometimes appropriate for interchange ramps, turning roadways, and intersection curb returns. Occasionally, they may be used on roadway mainlines, but only where some physical control prevents the use of a simple curve. Where a compound curve is used on a roadway mainline, the radius of the flatter circular arc (R_1) should not be more than 50% greater than the radius of the sharper circular arc (R_2). In other words, $R_1 \leq 1.5\ R_2$.

3-3.02 Traveled Way Widening

Traveled way widening may be necessary on the inside edge of horizontal curves for the following reasons:

- Vehicles (especially trucks) occupy a greater effective width because rear wheels track inside of front wheels when rounding a curve.
- Where lane widths are less than 12 feet, drivers may experience difficulty in steering within the traffic lane on curves.

The designer should check the design vehicle path to determine if traveled way widening is necessary.

3-3.03 Station Equations

Departures from the line surveyed in the field usually involve changes in the length of alignment. To avoid revising the stationing throughout the project, station equations are introduced. The equation identifies two station numbers — one that is correct when measuring on the line back of the equation, and one that is correct when measuring on the line ahead of the equation.

For divided roadway projects where the base lines are not common, the tangent sections of the base lines should have stationing that is equal and opposite. Typically, the alignment should include a station equation at the PC or PT of horizontal curves, but not at both the PC and PT.

Typical station equations are illustrated in Figure 3-3-B.

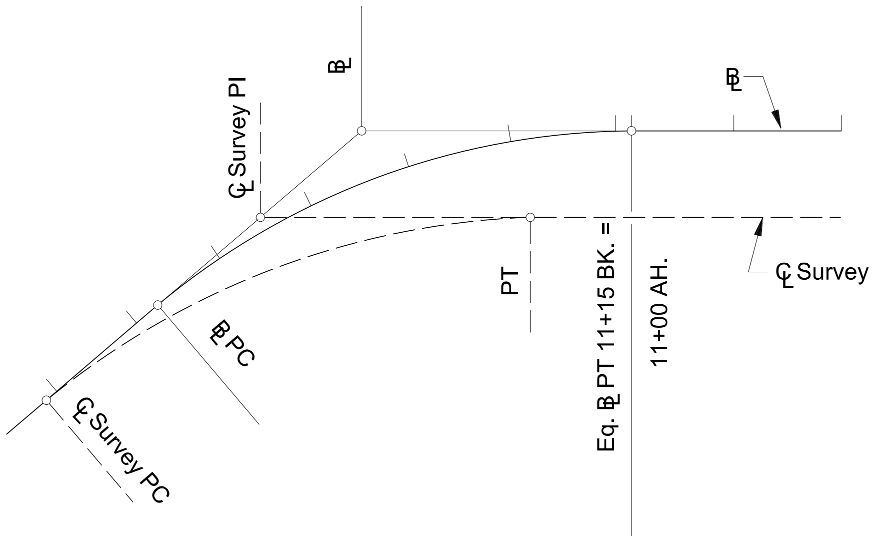
3-3.04 Divided Roadways

Where the median width of a divided roadway is a constant distance of 64 feet or less, the stationing and all other alignment computations should be based on a single survey baseline, which is normally the center of the median. A common plan grade and one set of curve data established from the base line should serve both roadways.

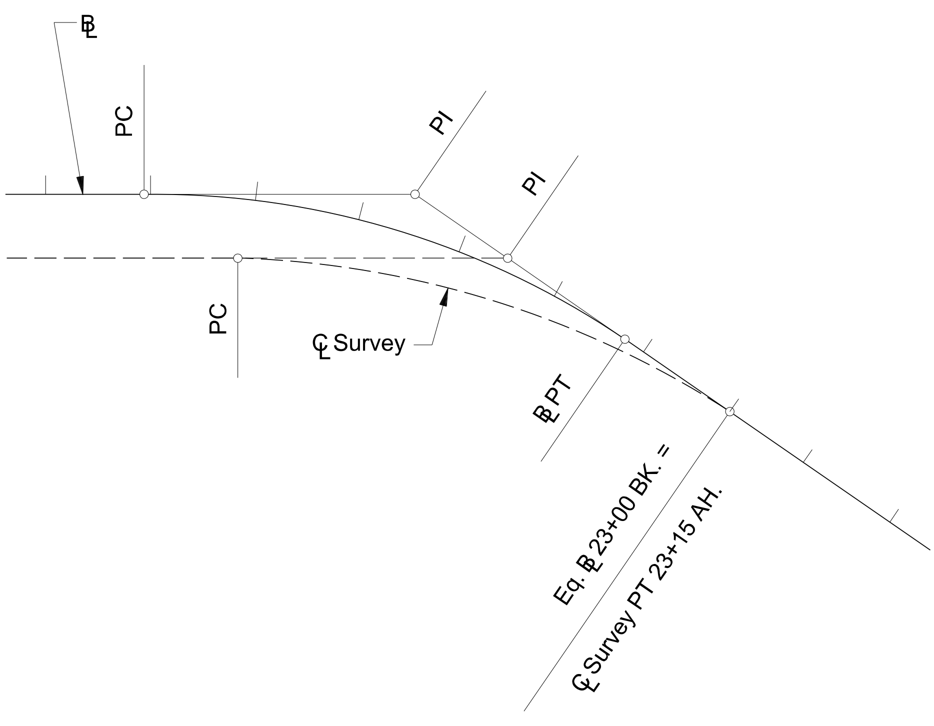
Where the median width on a divided roadway exceeds 64 feet, each roadway may have separate horizontal and vertical controls independent of a single survey base line.

3-3.04.01 Parallel Curves

The most common practice for horizontal curvature on divided roadways is the use of parallel curves (common center on radii) as illustrated in Figure 3-3-C. The deflection angle (Δ) and PC stationing are typically identical for the survey line, inside roadway, and outside roadway. The remaining curve data, including R, L, and T, vary for each alignment. Station equations typically re-establish common stationing from the PT on both roadways as discussed in Section 3-3.03.

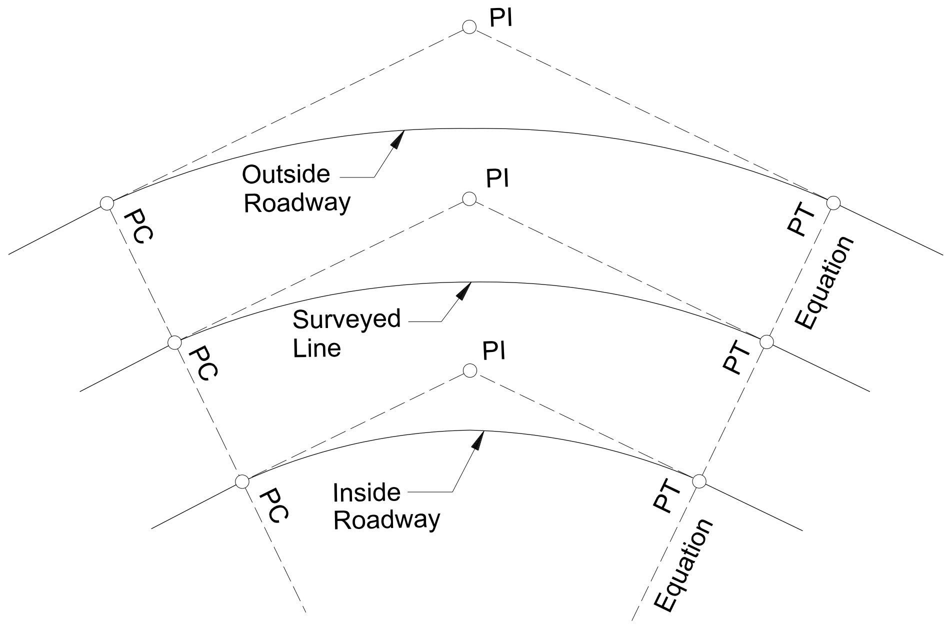


Projected Line Departs From Surveyed Line



Projected Line Returns To Surveyed Line

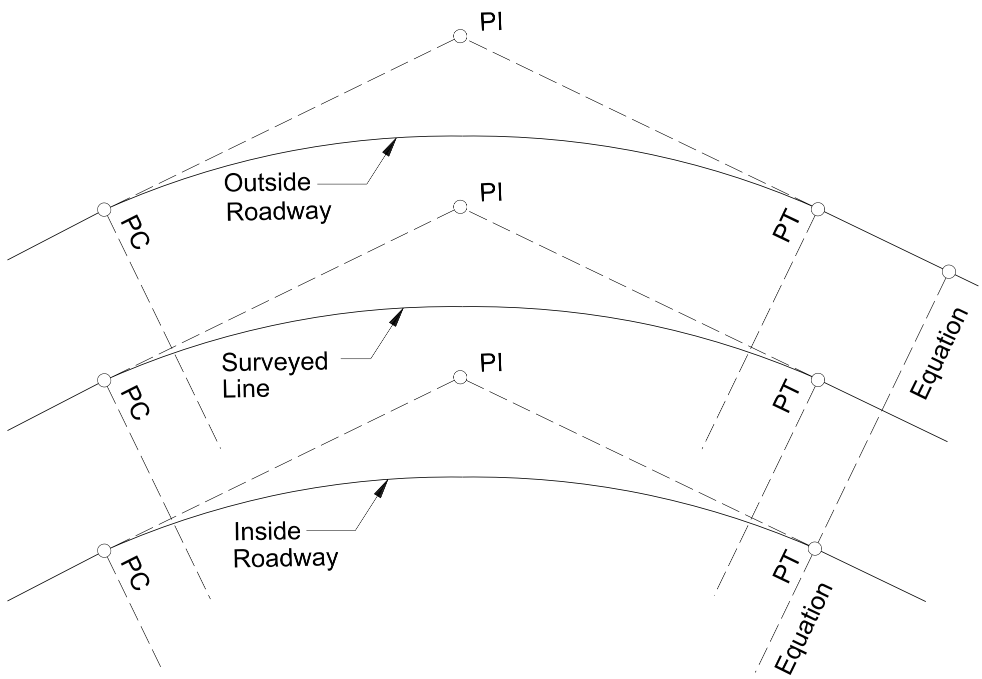
STATION EQUATIONS
Figure 3-3-B



PARALLEL CURVES
Figure 3-3-C

3-3.04.02 Non-Parallel Curves

Non-parallel curves may be used where field conditions warrant, as illustrated in Figure 3-3-D. The basic curve data, including Δ, L, and T, are typically identical for all three alignments. Stationing of the PC, PI, and PT usually differ for each line. Typically, an equation for stationing should be placed at the PT of the inside roadway to re-establish common stationing from that point ahead. However, for the outside roadway, an equation should be placed opposite the PT of the inside roadway to re-establish common stationing for the outside roadway.



NON-PARALLEL CURVES
Figure 3-3-D

3-4.0 SUPERELEVATION

The superelevation design tables in the *Standard Drawings* provide the following information:

1. superelevation rate (discussed in Section 3-4.01)
2. superelevation runoff length (discussed in Section 3-4.02)
3. impacts of the axis of rotation (discussed in Section 3-4.03)
4. range of allowable radii for each design speed and design superelevation (e_d)

The following subsections apply to rural roadways (all design speeds) and urban facilities ($V \geq 55$ miles per hour). Superelevation for other urban roadways is discussed in Section 14-2.04.

3-4.01 Superelevation Rate

The Department has adopted an e_{max} = 10% for the design of rural roadways (all design speeds) and urban roadways ($V \geq 55$ miles per hour). For urban roadways with a design speed of 50 miles per hour, an e_{max} = 6% should be used. For low-speed urban roadways ($V \leq 45$ miles per hour) an e_{max} = 4% should be used.

The *Standard Drawings* provide applicable radii for various combinations of superelevation rates for each design speed. Flatter curvature should be provided where the design radius will not provide the recommended SSD for horizontal curves. See Section 3-5.0. The indicated superelevation rates are applicable regardless of the number of lanes. Table 3-4-A provides the minimum radii for a normal crown slope and a reverse crown slope at various design speeds. Above the minimum radii for reverse crown, the radii and superelevation rates in the *Standard Drawings* should be used.

Table 3-4-A
MINIMUM RADII FOR NORMAL CROWN AND REVERSE CROWN (2% Typical)

Design Speed (mph)	Minimum Radius (ft)		
	Normal Crown	Reverse Crown	See *Standard Drawings*
30	R ≥ 3,320	3,320 > R ≥ 2440	R < 2,440
40	R ≥ 5,520	5,520 > R ≥ 4080	R < 4,080
50	R ≥ 8,280	8,280 > R ≥ 6130	R < 6,130
55	R ≥ 9,890	9,890 > R ≥ 7330	R < 7,330
60	R ≥ 11,700	11,700 > R ≥ 8630	R < 8,630
65	R ≥ 13,100	13,100 > R ≥ 9720	R < 9,720
70	R ≥ 14,700	14,700 > R ≥ 10,900	R < 10,900

Note: e_{max} = 10%

3-4.02 Superelevation Transition Length

The superelevation transition length is the distance required to transition the roadway from a normal crown slope to the full design superelevation rate (e_d). Superelevation should be introduced and removed uniformly over a length adequate for a given design speed. The total transition length is a function of the amount of superelevation, rate of transition, and width of rotation.

3-4.02.01 Segments

The total superelevation transition length consists of two segments — length of tangent runout and length of superelevation runoff.

- Tangent Runout (L_t) – Tangent runout is the distance needed to change from a normal crown section to a point where the adverse cross slope of the outside lane or lanes is removed and that portion of the cross section is level.

- Superelevation Runoff (L_r) – Superelevation runoff is the distance needed to change the cross slope from the end of the tangent runout (adverse cross slope removed) to a section that is sloped at the full design superelevation rate (e_d).

3-4.02.02 Tangent Runout

At the outer limits of the superelevation runoff sections, a tangent runout distance should be provided that is based on design speed and number of lanes rotated as shown in the *Standard Drawings.*

3-4.02.03 Superelevation Runoff

The *Standard Drawings* provide the superelevation runoff lengths (L_r) for 2-lane and 4-lane roadways, respectively, for various combinations of curve radii and design speeds. For 4-lane facilities with the median width greater than 64 feet, each roadway is evaluated separately. Runoff lengths are calculated using the following equation:

$$L_r = \frac{(w\ n_1)e_d}{\Delta}(b_w) \qquad \text{(Equation 3-4-1)}$$

Where:

L_r	=	minimum length of superelevation runoff (feet)
w	=	width of one traffic lane (feet) (typically 12 feet)
n_1	=	number of lanes rotated (see Table 3-4-B)
e_d	=	full design superelevation rate (%)
b_w	=	adjustment factor for number of lanes rotated (see Table 3-4-B)
Δ	=	maximum relative gradient, % (see Table 3-4-C)

The number of lanes rotated should be rounded to the nearest 0.5-lane increment and then the applicable row in Table 3-4-B should be used. For example, for a 5-lane section with four 11-foot travel lanes and a 14-foot Continuous Two-Way Left-Turn Lane (CTWLTL) rotated about the center line, the number of lanes rotated would be 2.5.

Table 3-4-B
ADJUSTMENT FACTOR FOR NUMBER OF LANES ROTATED

Number of Lanes Rotated	Number of Lanes Rotated (n_1)	Adjust Factor (b_w)	Length Increase Relative to One Lane Rotated (n_1 x b_w)
2	1	1.00	1.00
3	1.5	0.83	1.25
4	2	0.75	1.50
5	2.5	0.70	1.75
6	3	0.67	2.00
7	3.5	0.64	2.25
8	4	0.625	2.50

Note: *Where the median width is greater than 64 feet, each roadway is evaluated separately. For example, if a 6-lane divided facility has a median width greater than 64 feet, the number of lanes in one direction (three) is used in the table to read b_w = 0.83.*

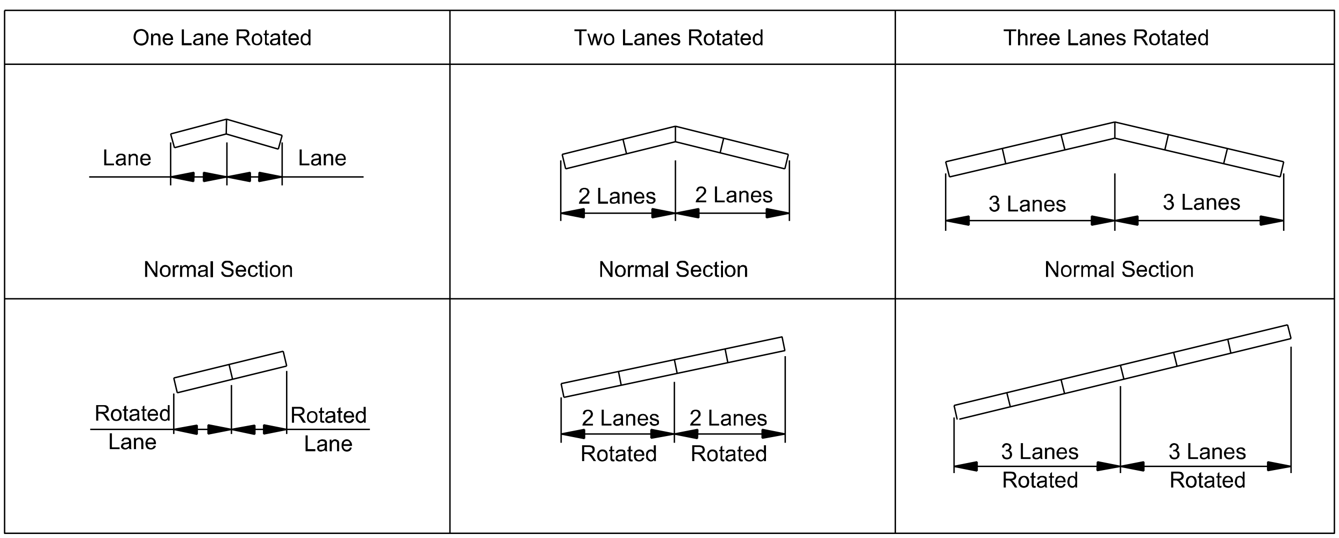


Table 3-4-C
RELATIVE LONGITUDINAL GRADIENTS

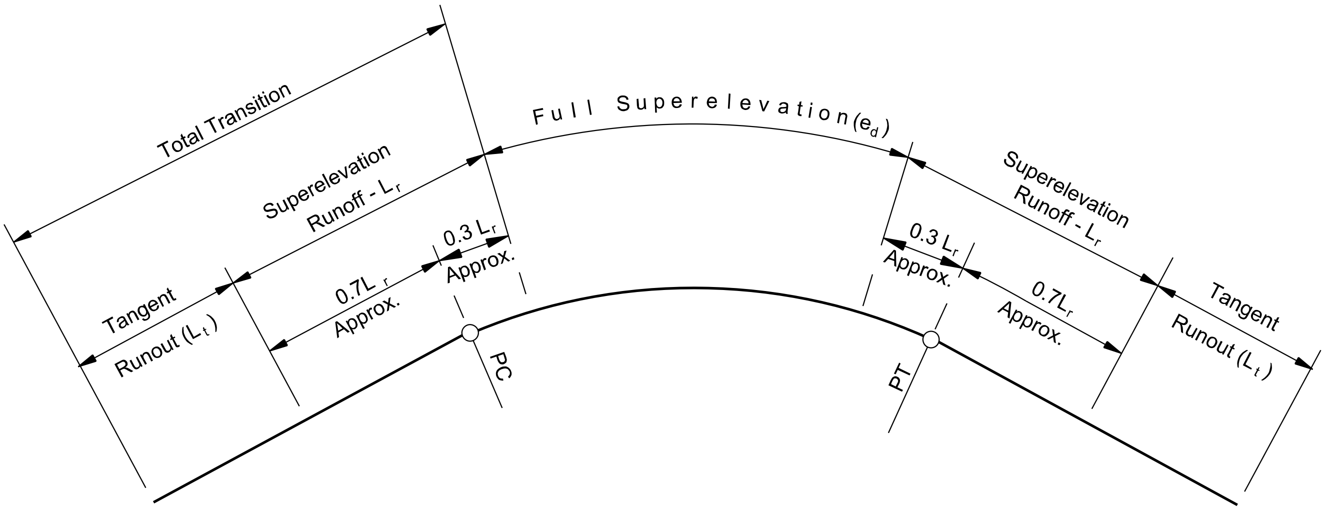
Design Speed (mph)	Maximum Relative Gradient (%) (Δ)	Equivalent Maximum Relative Slope
30	0.66	1:152
35	0.62	1:161
40	0.58	1:172
45	0.54	1:185
50	0.50	1:200
55	0.47	1:213
60	0.45	1:222
65	0.43	1:233
70	0.40	1:250

3-4.02.04 Application of Transition Length

Typical application of the superelevation transition length is shown in Figure 3-4-A for a simple curve.

On simple curves, approximately 70% of the superelevation runoff length should occur along the tangent section (before the PC and after the PT), and approximately 30% of the superelevation runoff length should occur within the circular curve. This method is a compromise between placing all of the transition on the tangent section (where superelevation is not needed) and placing all of the transition within the curve (where full superelevation is desired throughout the entire length). These values may be adjusted slightly to make the runoff coincide with 25-foot station increments or to avoid beginning or ending the transition within a bridge. When adjustments are made and the superelevation points are not as shown in the *Standard Drawings,* the adjusted superelevation points should be included in the plans. If a superelevation transition is proposed on a bridge, the Roadway Design Division should coordinate with the Bridge Design Division to determine an acceptable design.

On spiral curves, the length of superelevation runoff is coincident with the length of spiral (TS to SC or CS to ST), and full superelevation is provided between SC and CS.



APPLICATION OF THE SUPERELEVATION TRANSITION LENGTH
Figure 3-4-A

3-4.03 Superelevation Axis of Rotation

The superelevation axis of rotation is the line about which the pavement is revolved to superelevate the roadway. The criteria in the following subsections should be used to determine the proper axis of rotation.

Angular breaks occur in the edge of traveled way at points where the transition begins and ends. These breaks should preferably be rounded by insertion of vertical curves. While specific criteria have not been established for the lengths of vertical curves for these breaks, a vertical curve with a length (in feet) equal to the design speed (in miles per hour) can be used as an approximate guide. Greater lengths should be considered where feasible.

Additional details for superelevation transitions are provided in the *Standard Drawings.*

3-4.03.01 Case I: Superelevation Rotated About the Centerline

The roadway is typically rotated about the centerline profile for 2-lane roadways and multilane undivided roadways. Rotation about the centerline profile is illustrated in the *Standard Drawings*. Similarly, on a 2-lane roadway with an auxiliary lane (e.g., turning lane), the axis of rotation should typically be the centerline of the two through lanes.

Rotating the roadway about the inside edge of traveled way may occasionally be preferable to meet local conditions where the lower edge profile is a major control, such as for drainage.

Where the median width on a divided roadway exceeds 64 feet, each roadway is typically rotated separately about its centerline profile. On a 6-lane divided highway (three lanes in each direction) with a median width greater than 64 feet, each roadway is typically rotated about the edge of the inside travel lane.

3-4.03.02 Case II: Superelevation Rotated About the Median Edge

On divided roadways separated by a median width of 64 feet or less, a significant elevation differential may occur between the median-edge shoulders of two roadways rotated independently. To minimize elevation differential, the roadway is typically rotated about the two median-edge profiles, and the median is held in a horizontal plane as illustrated in the *Standard Drawings*. The plan grade, however, should desirably remain at the centerline of each roadway in such cases. If the plan grade is proposed at the median edge, the designer should clearly show the plan grade in the plans, and ensure the design is coordinated with any adjacent projects.

Where a divided roadway is separated by a concrete median barrier, consideration should be given to rotating about the two barrier edge profiles. The concrete barrier may be held in a horizontal plane and the shoulders adjacent to the barrier superelevated at the same rate as the traveled way.

3-4.04 Shoulder Superelevation

The algebraic difference between the traveled way slope and shoulder slope is known as the cross-slope rollover, and a maximum 7.0% cross-slope rollover is allowed. For example, where the superelevation rate for the outside travel lane equals 6.0%, the adjacent shoulder should slope away from the travel lane at a rate of 1.0%. Where the superelevation rate for the outside travel lane equals 10.0%, the adjacent shoulder should slope toward the travel lane at a maximum rate of 3.0%. The *Standard Drawings* illustrate the design details for shoulder superelevation within the superelevation transition length.

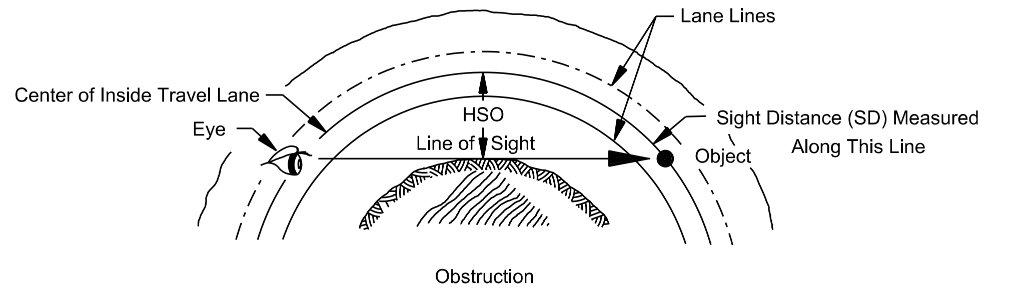
3-5.0 HORIZONTAL SIGHT DISTANCE

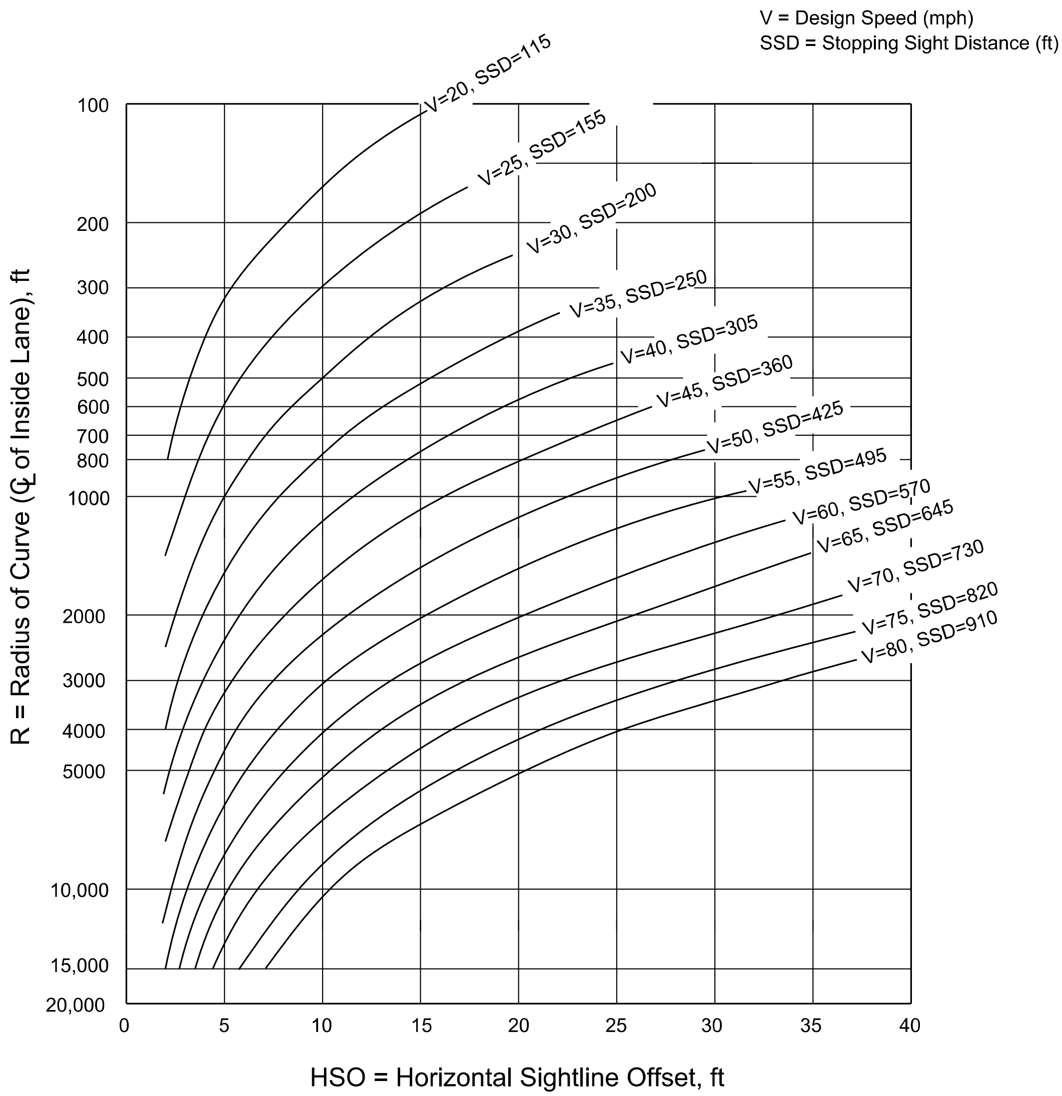
Sight obstructions on the inside of a horizontal curve are obstacles of considerable length that interfere with the line of sight on a continuous basis. Examples of such obstructions include walls, median barriers, bridge rails, cut slopes, wooded areas, buildings, and tall farm crops. In general, point obstacles (e.g., traffic signs, utility poles) are not considered sight obstructions. Each curve should be examined individually to determine whether it is necessary to remove an obstruction or adjust the horizontal alignment to obtain the minimum sight distance.

Sight obstructions may also interfere with Intersection Sight Distance (ISD). Chapter 6, “At-Grade Intersections”, and Chapter 7, “Grade Separations and Interchanges”, should be reviewed for sight distance recommendations at intersections and interchanges, respectively.

3-5.01 Horizontal Sight Line Offset

A horizontal sight line offset (HSO) should be provided between the center of the inside travel lane and the sight obstruction. Figure 3-5-A provides the criteria to determine the HSO at various design speeds for SSD values. See Table 2-9-A for sight distance values. For application, the eye height is 3.5 feet and the object height is 2.0 feet. The line-of-sight intercept with the obstruction is at the midpoint of the sight line and 2.75 feet above the center of the inside travel lane.





HORIZONTAL SIGHT LINE OFFSET (HSO)
(Length of Curve > Sight Distance)
Figure 3-5-A

The HSO values from Figure 3-5-A only apply to that portion of the curve between the PC and the PT. In addition, the figure does not apply to curves whose lengths are less than the design sight distance. The following subsections describe how to determine HSO for cases where the length of curve is greater than the sight distance and for cases where the length of curve is less than the sight distance.

3-5.01.01 Length of Curve Greater Than Sight Distance

Where the length of curve (L) is greater than the sight distance (SD) (stopping, passing, decision) used for design, the following equation should be used to calculate the minimum clearance on the inside of the horizontal curve:

$$HSO = R\left(1 - \cos\left[\frac{28.65SD}{R}\right]\right) \qquad \text{(Equation 3-5-1)}$$

Where:

HSO	=	Horizontal sight offset from the center of the inside travel lane to the obstruction (feet)
R	=	Radius of curve (feet)
SD	=	Sight distance (feet)

The HSO values from Equation 3-5-1 and Figure 3-5-A apply between the PC and PT. In addition, some transition is necessary in advance of and beyond the horizontal curve. The following steps should be used:

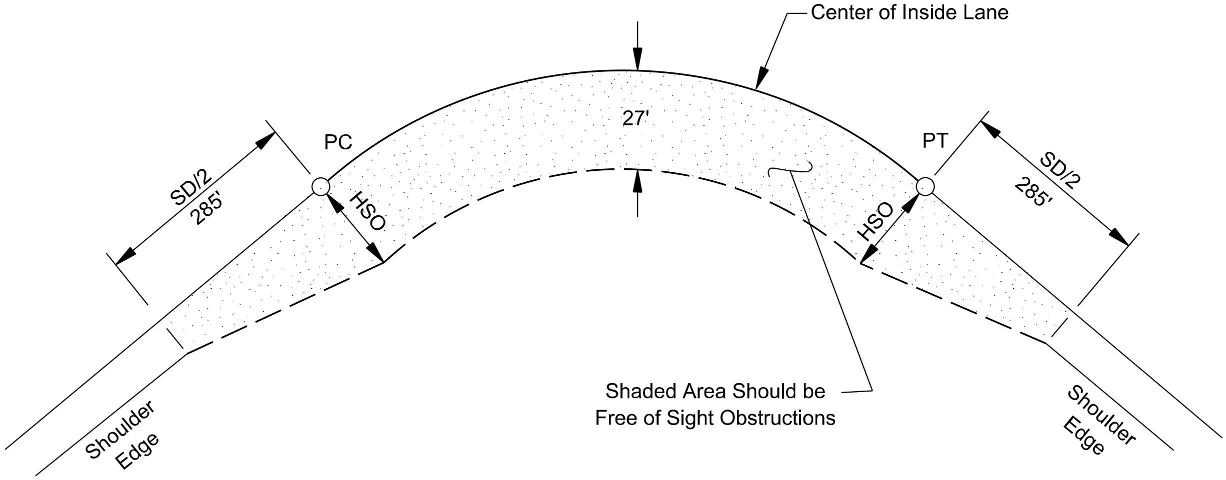
Step 1: A point that is on the outside edge of shoulder and a distance of SD/2 before the PC should be located.

Step 2: A point that is a distance HSO measured laterally from the center of the inside travel lane at the PC should be located.

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

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

The example in Figure 3-5-B illustrates the determination of the minimum clearance before and after a horizontal curve.



Example 3-5-1

Given: Design Speed = 60 miles per hour
R = 1500 feet
L = 600 feet

Problem: Determine the minimum horizontal clearance sightline offset for a horizontal curve on a 2-lane roadway assuming passenger car SSD.

Solution: Table 2-9-A yields an SSD = 570 feet for 60 miles per hour. Using Equation 3-5-1 for horizontal clearance (L > SD):

$$HSO = 1500\left(1 - \cos\left[\frac{(28.65)\ (570)}{1500}\right]\right) = 27.0 \text{ feet}$$

This answer is verified by Figure 3-5-A.

The above figure also illustrates the minimum clearance before and after the horizontal curve.

MINIMUM SIGHT CLEARANCE FOR HORIZONTAL CURVES
(L > SSD)
Figure 3-5-B

3-5.01.02 Length of Curve Less Than Sight Distance

Where the length of curve is less than the design sight distance, the HSO value from Equation 3-5-1 will never be reached. As an approximation, the horizontal clearance for these curves should be determined as follows:

Step 1: For the given R and SD, HSO should be calculated assuming L > SD.

Step 2: The maximum HSO′ value should be used at a point of L/2 beyond the PC. HSO′ is calculated using the following proportion:

$$\frac{HSO'}{HSO} = \frac{1.2\ L}{SD} \qquad \text{(Equation 3-5-1)}$$

$$HSO' = \frac{1.2\ (L)(HSO)}{SD} \qquad \text{(Equation 3-5-2)}$$

Where: $HSO' \leq HSO$

Step 3: A point should be located that is on the outside edge of shoulder and a distance of SD/2 before the PC.

Step 4: The two points located in Steps 2 and 3 should be connected. The area between this line and the roadway should be clear of all continuous obstructions.

Step 5: A symmetrical application of Steps 2 through 4 should be provided on the exiting portion of curve.

The example in Figure 3-5-C illustrates the determination of the minimum clearance where L < SD.

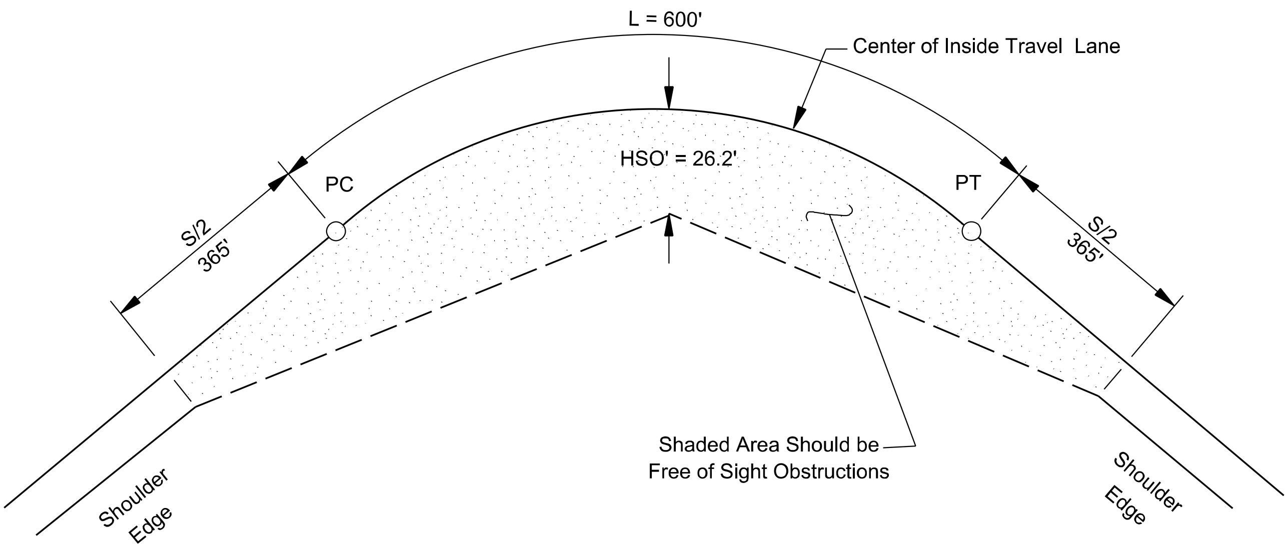
3-5.01.03 Longitudinal Barriers

Longitudinal barriers (guardrail, concrete barriers, bridge rails, etc.) may obstruct sight distance in a horizontal curve when such barriers are placed along or relatively close to the traveled way. Project designs along new alignments should include horizontal curves that meet the minimum horizontal sight distance. Horizontal sight distances along new alignments can, however, be impacted by factors other than sight distance considerations where the resulting horizontal sight distances are below the established minimums. In many such locations, there are often no feasible alternatives to the barrier locations.

Maintaining minimum horizontal sight distances along longitudinal barrier installations may not be feasible for projects along existing roadways. On such projects, the existing horizontal sight distances will typically be acceptable unless a crash pattern associated with an existing horizontal sight distance exists. If a crash pattern does exist, consideration should be given to realigning the roadway. See Chapter 12, “Existing Roadways”, for more information on projects on existing roadways.

3-5.02 Decision Sight Distance

Table 4-5-B provides minimum values for decision sight distance (DSD) based on design speed. If DSD is used at a horizontal curve, the applicable values should be used in Equation 3-5-1 to determine the minimum clearance on a horizontal curve.



Example 3-5-2

Given: Design Speed = 70 miles per hour
R = 2,500 feet
L = 600 feet

Problem: Determine the minimum horizontal sightline offset for the horizontal curve on a 2-lane roadway assuming SSD.

Solution: Table 2-9-A yields an SSD of 730 feet for 70 miles per hour. Therefore, L < SD (600 feet < 730 feet), and the horizontal clearance is calculated from Equation 3-5-1 and 3-5-2 as follows:

$$HSO\left(L > SD\right) = 2500\left[1 - \cos\frac{(28.65)(730)}{2500}\right] = 26.6 \text{ feet}$$

$$HSO' = \left(L < SD\right) = \frac{1.2(600)(26.6)}{730}$$

$$HSO' = 26.2 \text{ feet}$$

A minimum clearance of 26.2 feet should be provided at a distance of L/2 = 300 feet beyond the PC. The obstruction-free triangle around the horizontal curve would be defined by HSO′ (26.2 feet) at L/2 and by points at the shoulder edge at SD/2 = 365 feet before the PC and beyond the PT.

MINIMUM SIGHT CLEARANCE FOR HORIZONTAL CURVES
(L < SSD)
Figure 3-5-C

3-5.03 <u>Passing Sight Distance</u>

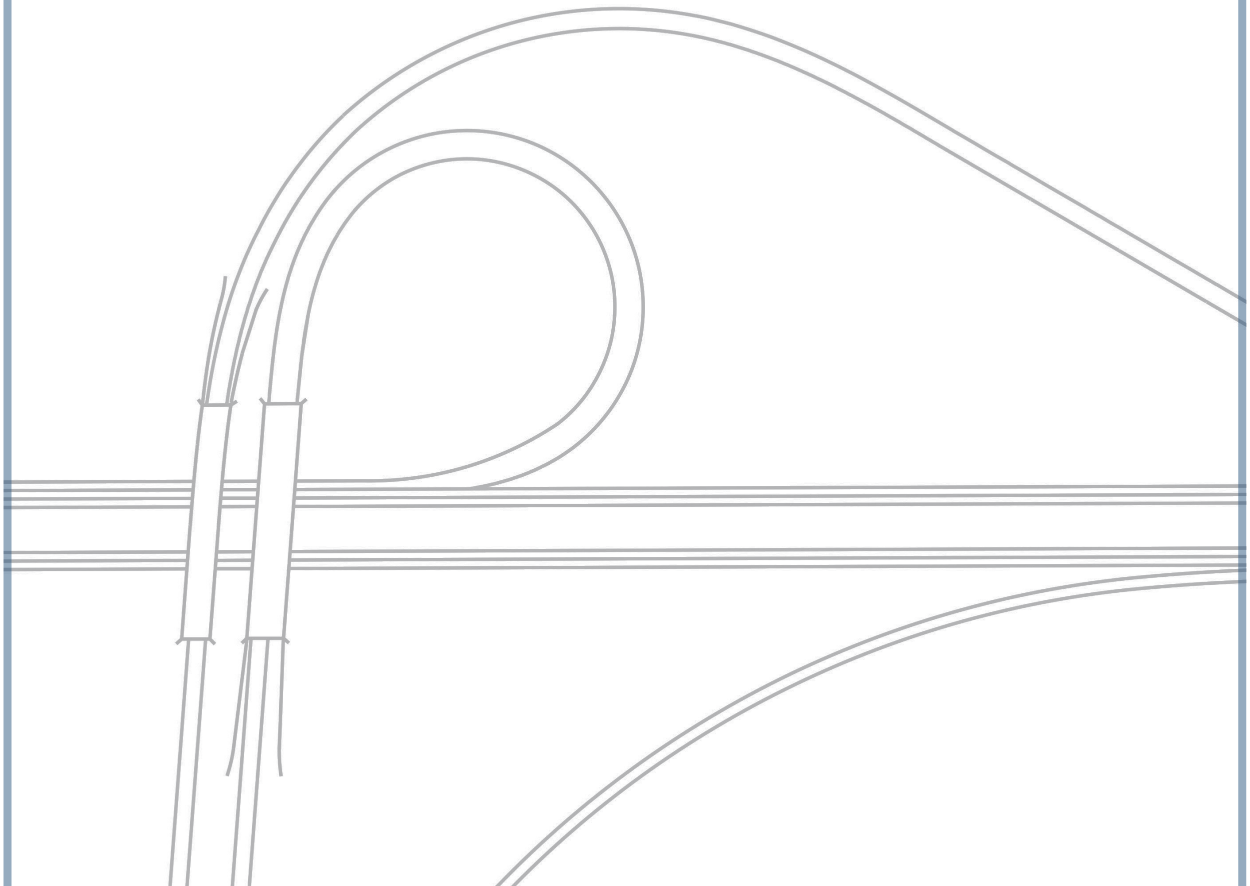
On high-volume, 2-lane roadways, passing sight distance may occasionally be warranted to maintain capacity. Table 4-5-C provides minimum passing sight distances based on design speed.

Generally, it is infeasible to provide passing sight distance on horizontal curves, which often yield very large HSOs for the recommended clearance. In addition, many drivers will not pass on horizontal curves regardless of the available sight distance. Where passing sight distance is used, Equation 3-5-1 should be used to calculate the minimum clearance.

3-6.0 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2018.

2. *Transition Curves for Highways*, Barnett.



CHAPTER 4

Vertical Alignment

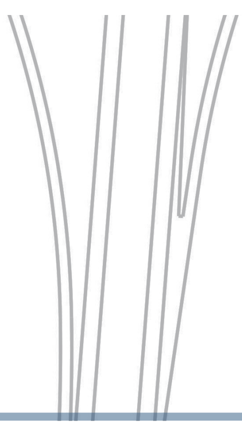


Table of Contents

Chapter 4
VERTICAL ALIGNMENT

Chapter 4 discusses the geometric design criteria for alignment coordination, grades, truck climbing lanes, vertical curves, and plan grade. Vertical alignment should be established by the judgments and decisions of the designer, which will affect the costs and safety characteristics of the facility.

4-1.0 GENERAL CRITERIA

4-1.01 Design Considerations

The following general criteria should be considered in the design of vertical alignment:

1. Consistency – A vertical alignment should desirably consist of smooth and gradual changes in a grade line that are consistent with both the roadway type and the character of terrain.

2. Broken-Back Grade – Broken-back curves (i.e., two crest or two sag vertical curves separated by a short tangent) should be avoided if possible. Desirably, a tangent distance of approximately 500 feet or more should be provided between two vertical curves, or one long vertical curve should be used instead.

3. Long Grades – On long grades, it is preferable to place the steepest grades at the bottom and flatter grades near the top of the ascent. Another option is to consider breaking the sustained grade by short intervals of flatter grade instead of providing a uniform grade that is only slightly below the recommended maximum.

4. Grades Through Intersections – Moderate to steep grades (> 3%) should preferably be reduced through intersections, which will benefit turning vehicles and minimize the potential for crashes.

5. Intersection Locations – Intersecting roadways should preferably not be located at or near the VPC or VPT of a crest vertical curve because such a combination could result in inadequate Intersection Sight Distance (ISD). See Section 4-2.0 for definitions of VPC and VPT.

6. Passing – Vertical sight distance restrictions may reduce passing opportunities. The designer should consider maximizing the passing opportunities on high-volume, 2-lane roadways to ensure adequate capacity. In extreme cases, it may be desirable to introduce passing lanes.

7. Terrain – The terrain designation has a significant impact on several of the geometric design elements (e.g., maximum allowable grades and design speed). The following general definitions apply:

a. Level – Sight distances are either long or could be made long without major construction expense.

b. Rolling – The natural slopes consistently rise above and fall below the roadway grade. Occasionally, steep slopes present some restriction to the desirable alignment.

c. Mountainous – Longitudinal and transverse changes in elevation are abrupt. Benching and side hill excavations are frequently necessary to provide a desirable alignment.

For design applications, Mississippi's terrain is considered either level or rolling.

8. Cut Sections – Sag vertical curves should preferably be avoided in cut sections, unless adequate drainage can be provided.

9. Earthwork – Optimizing the earthwork should be considered when setting the vertical alignment in order to reduce the amount of borrow or excess material. For guidance on balancing earthwork, see Section 4-7.03.

4-1.02 Vertical and Horizontal Coordination

The designer should check the coordination of the vertical and horizontal alignments by considering the following:

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

2. Coordination – Vertical curvature superimposed upon horizontal curvature (i.e., vertical and horizontal PIs at approximately the same stations) generally provides a more pleasing appearance and can reduce the number of sight distance restrictions. Successive changes in profile, not in combination with horizontal curvature, may result in an aesthetically unpleasing design.

3. Crest Vertical Curves – Sharp horizontal curvature should preferably not be introduced at or near the top of pronounced crest vertical curves because the driver may not be able to perceive the horizontal change in alignment, especially at night when headlight beams project straight ahead into space. This situation can be avoided if the horizontal curvature leads the vertical curvature or if design values exceed the minimums.

4. Sag Vertical Curves –Sharp horizontal curves should preferably not be introduced at or near the low point of pronounced sag vertical curves or at the bottom of steep grades. Because visibility to the road ahead can be foreshortened, any horizontal curvature with a radius that is at or near the minimum radius can create a distorted appearance. At the bottom of long grades, vehicular speeds may be higher, particularly for trucks, and erratic operations may occur, especially at night and during icy conditions.

5. Superelevation Transitions – Flat areas that hinder proper drainage should be avoided where feasible, especially where superelevation transitions coincide with the top of crests, bottom of sags, or relatively flat tangent grade sections.

6. Intersections – At intersections, horizontal and vertical alignment should provide a design that produces sufficient ISD and gradients for vehicles to slow, stop, or turn. See Chapter 6, “At-Grade Intersections.”

7. Divided Roadways – On divided facilities with wide medians, it may be advantageous to provide independent alignments and profiles for the two 1-way roadways. Where traffic volumes justify a divided facility, and where rolling or rugged terrain exists, a better design can result from the use of independent alignments and profiles.

4-2.0 DEFINITIONS

Figure 4-2-A illustrates the various elements of vertical alignment. The following definitions apply to each of the elements shown in Figure 4-2-A:

1. Vertical Point of Curvature (VPC) – The point at which the tangent grade ends and the vertical curve begins.

2. Vertical Point of Tangency (VPT) – The point at which the vertical curve ends and the tangent grade begins.

3. Vertical Point of Intersection (VPI) – The point where the extension of two tangent grades intersect.

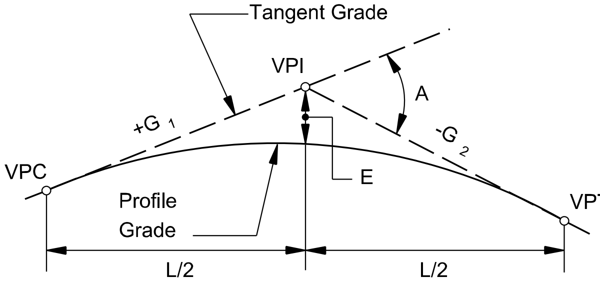
4. Grade Slopes (G_1 or G_2) – The rate of slope between two adjacent VPIs expressed as a percent. The numerical value for percent of grade is the vertical rise or fall in feet for each 100 feet of horizontal distance.

5. Sign Convention – Upgrades in the direction of stationing are identified as plus (+). Downgrades are identified as minus (–).

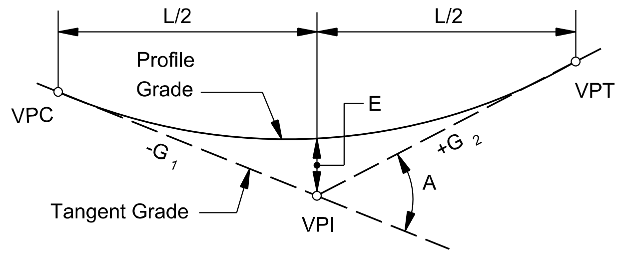
6. External Distance (E) – The vertical distance between the VPI and the roadway surface along the vertical curve.

7. Algebraic Difference (A) – The value of A is determined by the deflection, in percent, between the two tangent grades.

8. Length of Vertical Curve (L) – The horizontal distance, in feet, from the VPC to the VPT.



Crest Vertical Curve



Sag Vertical Curve

SCHEMATIC OF VERTICAL CURVES
Figure 4-2-A

4-3.0 GRADES

The vertical alignment of a roadway is referenced to a plan grade line, which consists of a series of tangents that are connected by vertical parabolic curves.

4-3.01 Maximum Grades

Chapters 2, 12, and 14 present the Department's criteria for maximum grades based on functional classification, urban/rural location, project scope of work, type of terrain, and design speed. Grades that are flatter than the maximum are desirable.

4-3.02 Minimum Grades

On roadways with curb and gutter, the minimum longitudinal gradient should be 0.2%; however, a grade of 0.4% or greater is desirable to facilitate longitudinal drainage.

On roadways without curb and gutter, level gradients are acceptable on pavements that are adequately crowned to drain laterally. However, except for the Mississippi Delta or other flat areas, a longitudinal grade of 0.4% or more is desirable because the original lateral crown slope may degrade as a result of swell, consolidation, maintenance operations, or resurfacing.

Combinations of superelevation transitions and flat grades should be avoided on bridges, as well as sag vertical curves that could result in a low point on a bridge. Although it is not always feasible to achieve, a longitudinal grade of 0.5% or more is desirable on bridges because flatter grades may not provide adequate bridge deck drainage. Roadway Design Division should coordinate with the Bridge Design Division for bridges where less than the desirable grade is proposed.

4-3.03 Critical Length of Grade

The upgrade gradient in combination with its full length should determine the resulting truck speed reduction. Figure 4-3-A provides the critical length of grade for a given percent grade and the corresponding truck speed reduction. This figure should be used for 2-lane roadways, and it is applicable to any design speed. For design, use of the 10 mile per hour speed reduction curve is recommended. If this value is exceeded, the designer should consider flattening the grade or should evaluate the need for a truck climbing lane (see Section 4-4.0).

Where an upgrade is preceded by a momentum grade (i.e., a downgrade), trucks often increase their speed to make the climb. Speed increases of five miles per hour on moderate downgrades (3% to 5%) and 10 miles per hour on steeper grades (6% to 8%) of sufficient length are reasonable adjustments. These values can be applied to Figure 4-3-A to allow the use of a higher speed reduction curve. If the critical length of grade is still exceeded, a capacity analysis may be advisable.

Examples 4-3-1 and 4-3-2 illustrate how to use Figure 4-3-A to determine the critical length of grade.

Example 4-3-1

Given: Level Approach
G = +4%
L = 1500 feet (length of grade)
Rural Arterial

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

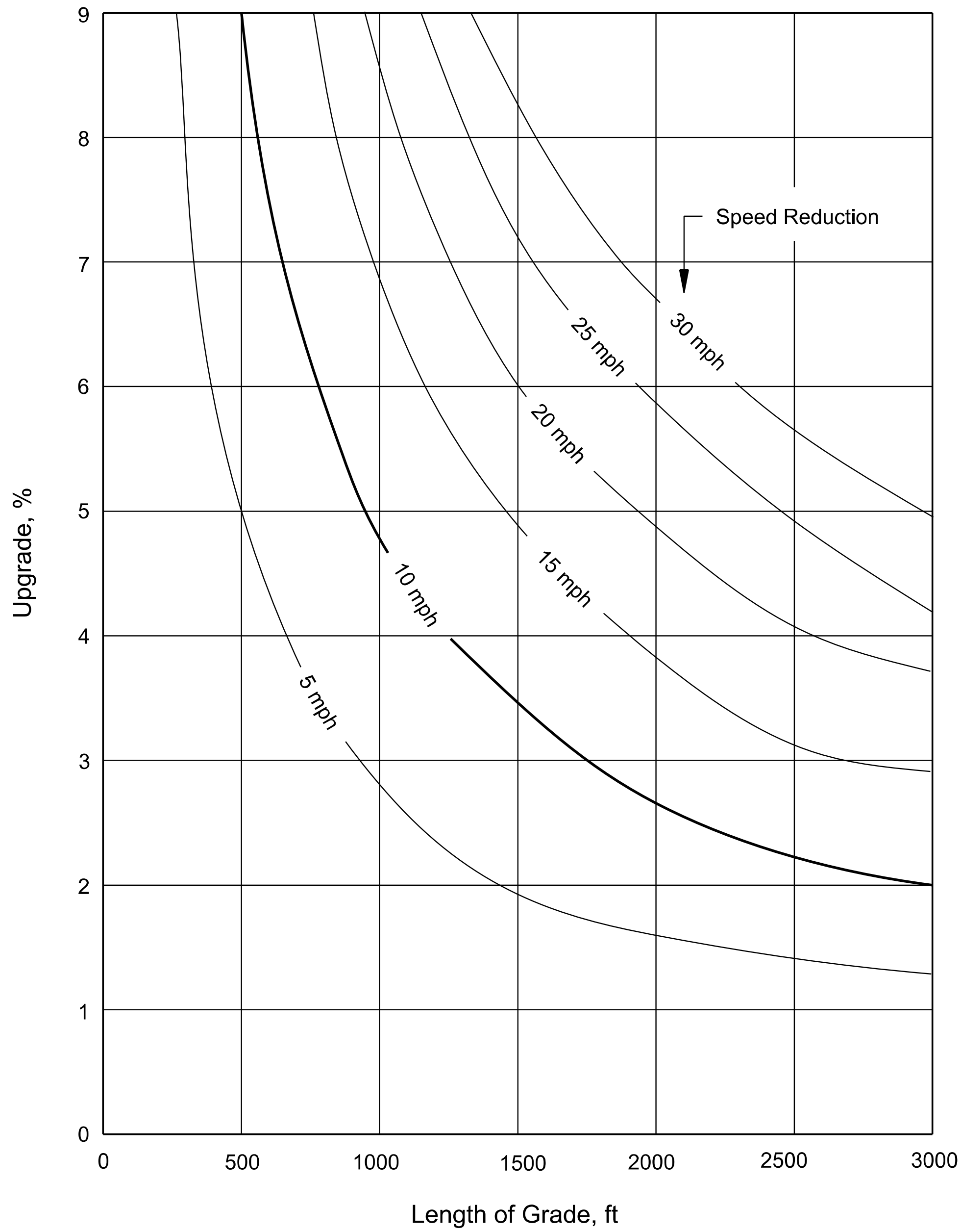
Solution: Figure 4-3-A yields a critical length of grade of 1200 feet for a 10 mile per hour speed reduction. The length of grade (L) exceeds this value. Therefore, flattening the grade should be considered, if feasible, or evaluating the need for a truck climbing lane should be studied.

Example 4-3-2

Given: Level Approach
G_1 = +4.5%
L_1 = 500 feet
G_2 = +2%
L_2 = 700 feet
Rural Arterial with a significant number of heavy trucks

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

Solution: From Figure 4-3-A, G_1 yields a truck speed reduction of five miles per hour. G_2 yields a speed reduction of approximately three miles per hour. The total of eight miles per hour is less than the maximum 10 mile per hour speed reduction. Therefore, the critical length of grade is not exceeded.



Notes:

1. *Typically, the 10 miles per hour curve should be used.*
2. *See examples in Section 4-3.03 for use of figure.*
3. *Figure is based on a truck with initial speed of 70 miles per hour. However, it may be used for any design or posted speed.*
4. *This figure is based on a 200 LB per horsepower heavy vehicle.*

CRITICAL LENGTH OF GRADE
Figure 4-3-A

4-4.0 TRUCK CLIMBING LANES

The following subsections discuss guidelines for determining the location, critical lengths of grade, and other design criteria for truck climbing lanes. Guidance is also provided on how to develop truck speed profiles. For additional guidance on these topics, see AASHTO's *A Policy on Geometric Design of Highways and Streets.*

4-4.01 <u>Location Guidelines</u>

A truck climbing lane may be necessary to allow a specific upgrade to operate at a desirable Level of Service (LOS).

4-4.01.01 2-Lane Roadways

A truck climbing lane should be considered on a 2-lane, 2-way roadway, if the following conditions apply:

1. The upgrade traffic flow is in excess of 200 vehicles/hour.
2. The heavy vehicle volume (i.e., trucks, buses, recreational vehicles) exceeds 20 vehicles/hour during the design hour.
3. One of the following conditions exists:
   a. The critical length of grade is exceeded for the 10 mile per hour speed reduction curve (see Figure 4-3-A).
   b. The LOS on the upgrade is E or F.
   c. There is a reduction of two or more LOS when moving from the approach segment to the upgrade.
   d. The construction costs and construction impacts of the additional lane (e.g., environmental, right of way) are considered reasonable.

In addition, high-crash frequencies may justify the addition of a truck climbing lane regardless of grade or traffic volumes.

4-4.01.02 Multilane Roadways

A truck climbing lane should be considered on a multilane roadway if the following conditions apply:

1. The LOS for the serviced direction and/or the upgrade is E or F.
2. The directional service volume exceeds 1000 vehicles/hour/lane.
3. One of the following conditions exists:
   a. The critical length of grade is exceeded for the 10 mile per hour speed reduction curve (see Figure 4-3-A).

b. The LOS is degraded by one or more letter values when moving from the approach segment to the upgrade.

c. The construction costs and construction impacts (e.g., environmental, right of way) of the additional lane are considered reasonable.

In addition, high-crash frequencies may justify the addition of a truck climbing lane regardless of grade or traffic volumes.

4-4.02 Capacity Analysis

When conducting a capacity analysis, a desired LOS should be selected. The geometric design criteria tables in Chapters 2, 12, and 14 present LOS criteria for each roadway class and urban/rural location. The *Highway Capacity Manual* should be referenced for guidance on conducting capacity analyses for climbing lanes on 2-lane and multilane roadways.

4-4.03 Design Criteria

Table 4-4-A presents some of the key geometric design criteria for the design of a truck climbing lane. The following additional information should also be used:

1. Design Speed – The design speed of the truck climbing lane should typically be equal to the design speed of the project or the posted speed limit, whichever is less. Consideration should be given to the effect a momentum grade could have on the entering speed. See Section 4-3.03 for additional information on momentum grades.

2. Cross Slope – On tangent sections, the truck climbing lane cross slope should be the same as that of the adjacent travel lane.

3. Superelevation – For horizontal curves, desirably the truck climbing lane should be superelevated at the same rate as the adjacent travel lane.

4. Performance Curves – Figure 4-4-A presents the deceleration and acceleration rates for a 200 LB per horsepower truck.

5. End of Full-Width Lane – In addition to the criteria in Table 4-4-A, sufficient sight distance should be available to the point where the truck, recreational vehicle, or bus begins to merge back into the through travel lane. At a minimum, the available sight distance should be 1.25 times the Stopping Sight Distance (SSD) based on a 2-foot object height. Desirably, the driver should have Decision Sight Distance (DSD) available to the roadway surface (i.e., height of object equals 0.0 feet) at the end of the taper.

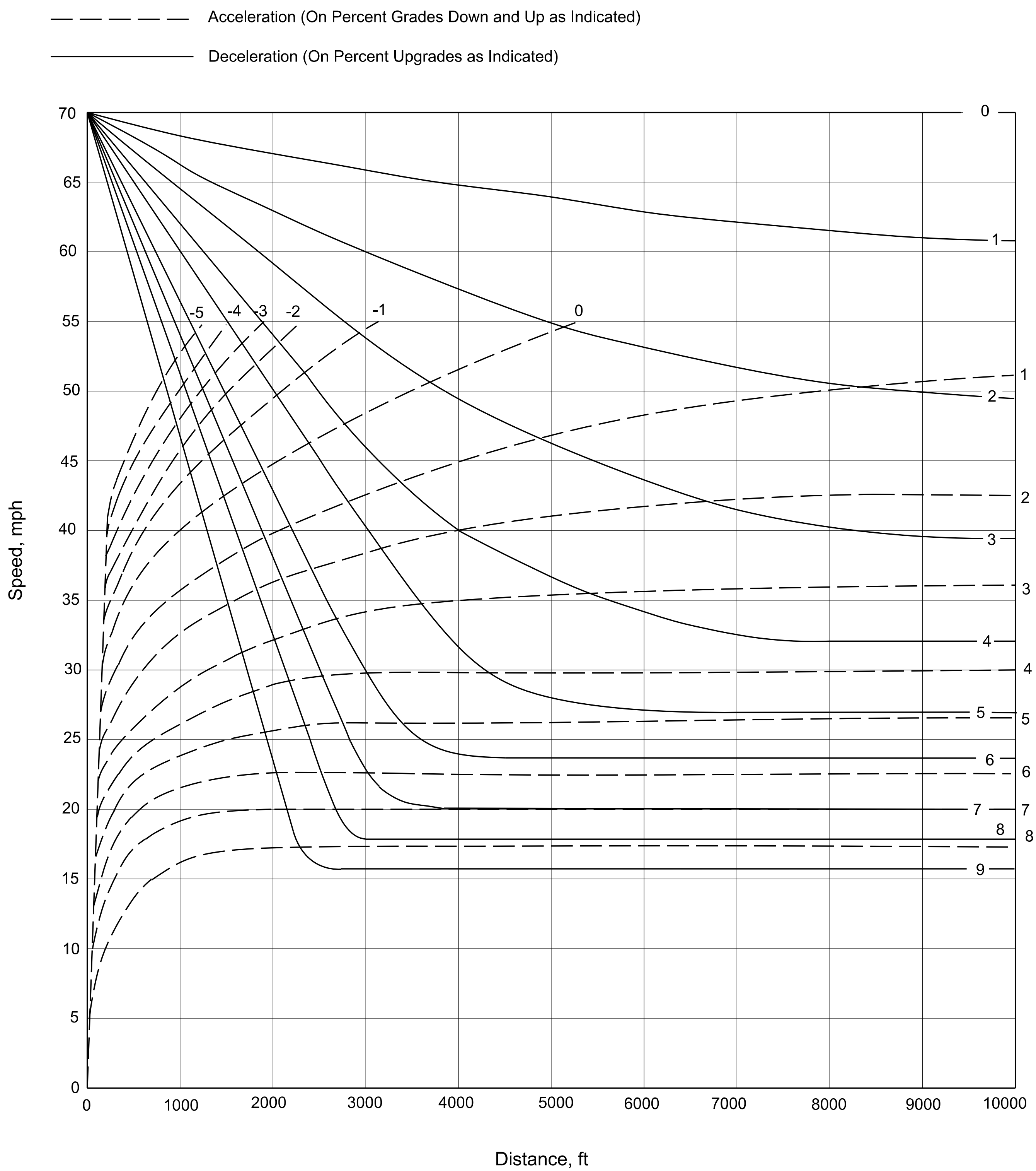
   The full-lane width should extend beyond the crest vertical curve and preferably not end on a horizontal curve.

6. Pavement Markings – Figure 4-4-B illustrates the design and pavement markings of a typical climbing lane.

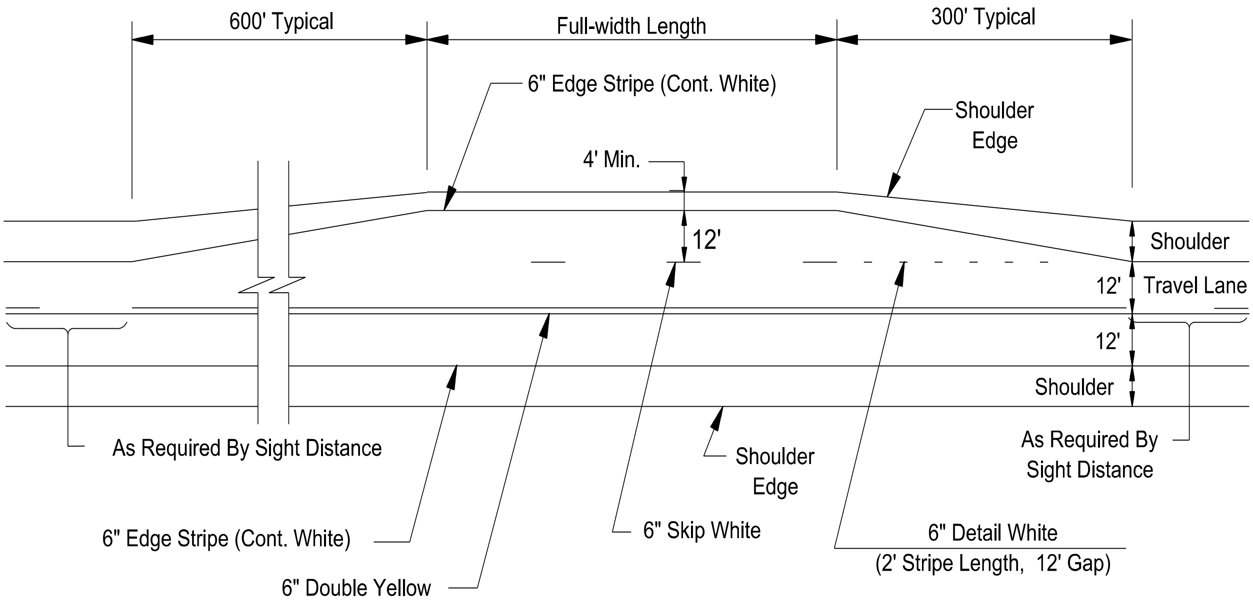
Table 4-4-A
DESIGN CRITERIA FOR TRUCK CLIMBING LANE

Design Element	Desirable	Minimum
Lane Width	12 ft	Width of adjacent lane
Shoulder Width	Same width as approach shoulder	4 ft
Cross Slope on Tangent	2%	2%
Beginning of Full-Width Lane (1)	Location where the truck speed has been reduced to 10 mph below the posted speed limit	Location where the truck speed has been reduced to 45 mph
End of Full-Width Lane (2)	Location where truck has reached the posted speed or 55 mph, whichever is less	Location where truck has reached 10 mph below the posted speed limit
Entering Taper	300 ft	150 ft
Exiting Taper	Freeway: 70:1 Other Roadways: 600 ft	50:1
Minimum Full-Width Length		1000 ft

Note: Figure 4-4-A should be used to determine truck acceleration and deceleration rates.



PERFORMANCE CURVES FOR TRUCKS
(200 lb/hp)
Figure 4-4-A



TYPICAL CLIMBING LANE
Figure 4-4-B

4-5.0 VERTICAL CURVES

4-5.01 Characteristics of Vertical Curves

Vertical curves have the properties of a simple parabolic curve. The vertical offset from the tangent at any point varies as the square of the horizontal distance from the end of the curve. Figure 4-5-A illustrates a symmetrical vertical curve. Curves that are offset below the tangents are called crest vertical curves; those that are offset above the tangents are sag vertical curves.

The rate of change of grade per foot (R) along a vertical curve is constant, and is measured by dividing the algebraic difference between the grades (A), in percent, by the length of the curve (L), in feet (i.e., A/L). This value gives the percent change in grade per horizontal foot of curve.

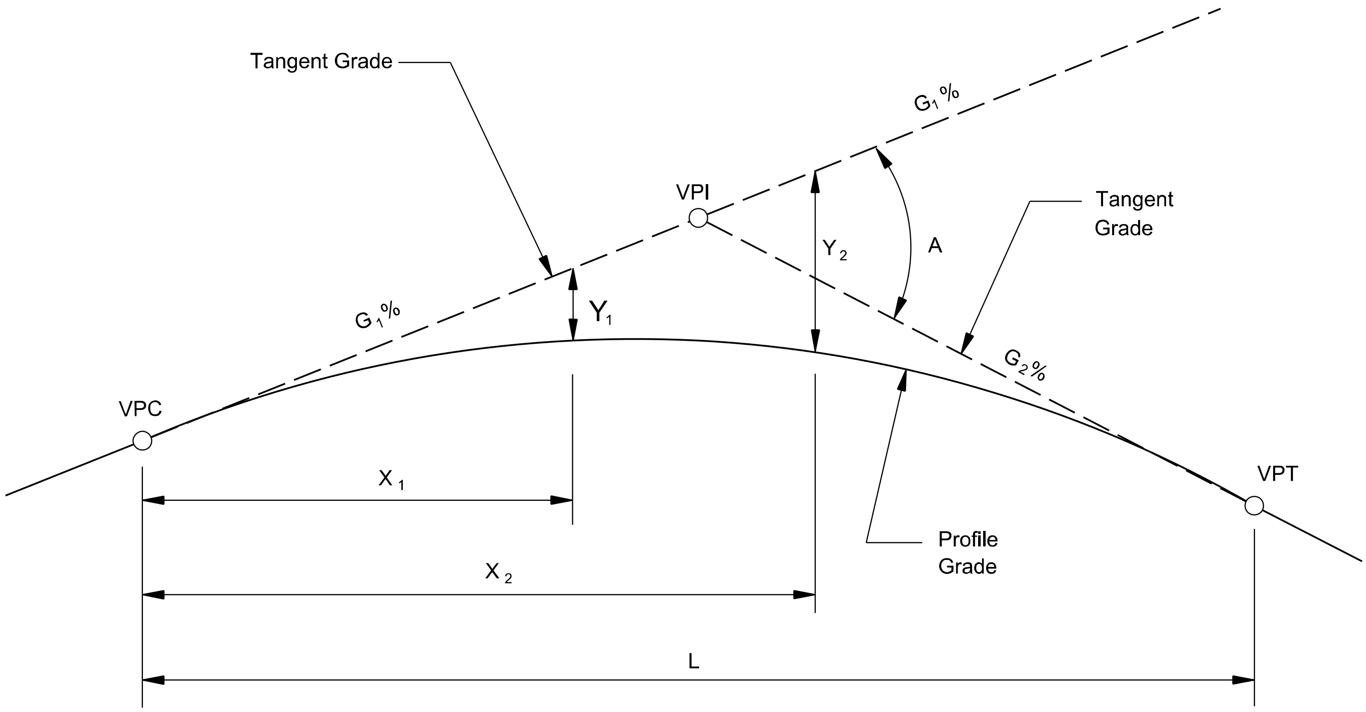
The reciprocal of this value (i.e., L/A) represents the horizontal distance required to produce a 1% change in the gradient along the curve. The expression L/A is designated K and is useful for determining minimum lengths of vertical curves. Based on the geometrics of each sight distance condition, formulas are used to compute values of K for each design speed. The minimum length of vertical curve is:

$$L = KA$$

Where:

L = Minimum length of vertical curve (feet)
A = Algebraic difference between tangent grades (percent)
K = a constant value for the design speed

Values of K are presented in the following subsections for crest and sag vertical curves.



The computation of an offset at any given point on the vertical curve is determined by:

$$Y = RX^2 / 200$$

Where:

Y	=	Vertical offset to the tangent grade from any given point on the vertical curve (feet)
X	=	Horizontal distance from VPC to any given point on the vertical curve (feet)
R	=	$A/L = [(G_2 – G_1)/L]$
	=	Rate of change of grade along the vertical curve (% per foot)
G_2	=	Tangent grade beyond the VPI (%)
G_1	=	Tangent grade before the VPI (%)
A	=	$(G_2 – G_1)$ = the algebraic difference between tangent grades (%)
L	=	Total length of vertical curve (feet)

The computation of the horizontal distance to the high point (crest) or low point (sag) of a vertical curve is determined by:

$$X_{high/low} = [LG_1 / (G_1 – G_2)]$$

SYMMETRICAL VERTICAL CURVE COMPUTATION
Figure 4-5-A

4-5.02 Crest Vertical Curves

The minimum length of a crest vertical curve in feet, regardless of sight distance, should be L_{min} equals 3V, where V equals the design speed in miles per hour.

4-5.02.01 Stopping Sight Distance

A driver should be able to see an object on the road in time to stop. The crest of a vertical curve should not obstruct the line of sight. A 3.5-foot eye height and a 2.0-foot object height are used in the equation for crest vertical curves. Table 4-5-A provides the criteria to determine the minimum length of a crest vertical curve at various design speeds and SSD values. Wherever feasible, longer SSDs should be considered.

Table 4-5-A
K-VALUES FOR CREST VERTICAL CURVES
(Stopping Sight Distance)

Design Speed (mph)	SSD Rounded for Design (ft)	K-Values Rounded for Design $K = S^2/2158$
30	200	19
35	250	29
40	305	44
45	360	61
50	425	84
55	495	114
60	570	151
65	645	193
70	730	247

When S is < L:

$$L = \frac{AS^2}{2158} = KA$$

When S is > L:

$$L = 2S - \frac{2158}{A}$$

Where:

S = *Sight distance for the design speed (feet)*
L = *Length of vertical curve (feet)*
A = *Algebraic difference between grades (%)*
K = *Horizontal distance required to produce a 1% change in gradient*

4-5.02.02 Decision Sight Distance

Table 4-5-B presents the minimum values for DSD. These distances provide the driver with more time to detect a hazard and decide what course of action to take. At some roadway locations, a crest vertical curve may obstruct the line of sight to an obstacle that may be unexpected or difficult to perceive. For example, an exit gore from a freeway may be located just beyond a crest vertical curve. Once the decision has been made to use DSD as a design control, the applicable DSD value should be used in the equation for crest vertical curves to yield the necessary curve length at the site. At a minimum, DSD should be calculated using a 3.5-foot eye height and a 2.0-foot object height. However, for some critical locations just beyond a crest vertical curve, the use of a 0.0-foot object height should be considered.

Table 4-5-B
DECISION SIGHT DISTANCE

Design Speed (mph)	Decision Sight Distance for Avoidance Maneuver (ft)				
	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

Avoidance Maneuvers are:

A: Stop on rural road
B: Stop on urban road
C: Speed/path/direction change on rural road
D: Speed/path/direction change on suburban road
E: Speed/path/direction change on urban road

4-5.02.03 Passing Sight Distance

On 2-lane roadways, the use of passing sight distance as a design control for crest vertical curves may be desirable. Table 4-5-C provides the criteria to determine the length of a crest vertical curve at various design speeds and minimum passing sight distance values. The eye height and object height are 3.5 feet.

Generally, it is infeasible to design crest vertical curves to provide for passing sight distance because of the high cost and the difficulty of fitting the resulting long vertical curves to the terrain. In addition, drivers may be reluctant to pass on vertical curves even where sufficient sight distance is available.

The Department's criteria for striping no-passing zones is presented in Section 10-1.01.

Table 4-5-C
K-VALUES FOR CREST VERTICAL CURVES
(Passing Sight Distance)

Design Speed (mph)	Passing Sight Distance Rounded for Design (ft)	K-Values Rounded for Design $K = S^2/2800$
30	500	89
35	550	108
40	600	129
45	700	175
50	800	229
55	900	289
60	1000	357
65	1100	432
70	1200	514

When S is < L:

$$L = \frac{AS^2}{2800} = KA$$

When S is > L:

$$L = 2S - \frac{2800}{A}$$

Where:

S = Sight distance for the design speed (feet)
L = Length of vertical curve (feet)
A = Algebraic difference between grades (%)
K = Horizontal distance required to produce a 1% change in gradient

4-5.03 Sag Vertical Curves

The minimum length of a sag vertical curve in feet, regardless of sight distance, should be $L_{min} = 3V$, where V equals the design speed in miles per hour.

4-5.03.01 Headlight Sight Distance

Nighttime driving conditions govern for sag vertical curves. The designated sight distance should be illuminated by the headlight beam with an assumed upward divergence of one degree. The height of the headlights is 2.0 feet and the object height is 0.0 inches (i.e., the pavement surface). Table 4-5-D provides the criteria to determine the length of a sag vertical curve at various design speeds and design SSD values.

4-5.03.02 Comfort Criteria

In lighted areas on urban roadways, the comfort criteria may be used for sag vertical curves, which are discussed in Chapter 14, "Geometric Design of Urban Roadways."

Table 4-5-D
K-VALUES FOR SAG VERTICAL CURVES
(Stopping Sight Distance)

Design Speed (mph)	SSD Rounded for Design (ft)	K-Values Rounded for Design $K = S^2/(400 + 3.5S)$
30	200	37
35	250	49
40	305	64
45	360	79
50	420	96
55	490	115
60	570	136
65	645	157
70	730	181

When S is < L:

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

When S is > L:

$$L = 2S - \frac{2158}{A}$$

Where:

S = Sight distance for the design speed (feet)
L = Length of vertical curve (feet)
A = Algebraic difference between grades (%)
K = Horizontal distance required to produce a 1% change in gradient

4-6.0 ESTABLISHING PLAN GRADE

The plan grade should represent the finished surface elevation at the centerline of the roadway or at the median edge of the pavement for divided roadways with a raised median. Manual computation procedures are described in the following subsections to ensure understanding of the concepts.

4-6.01 Grade Computations

Initial efforts for establishing the plan grade may be recorded on a working profile. Using the criteria in this manual, the designer should plot a tentative grade line on the working profile with the stationing and approximate elevation of each VPI clearly defined.

For ease of computation, it is desirable to locate VPIs at multiples of 25 feet within the stationing (e.g., Station 5 + 75). VPI elevations should be recorded to the nearest 0.01 foot. Once the VPIs are selected, GEOPAK or OpenRoads will calculate the grades.

4-6.02 Vertical Curve Computations

After the tangent grades have been established, the appropriate vertical curves should be designed for each VPI according to the criteria for sight distance and curvature.

Vertical curve plan grade elevations should be established by computing vertical offset distances between the tangent grade and the vertical curve at various points along the stationing. For crest vertical curves, these offset distances are subtracted from the tangent grade elevations. For sag vertical curves, the computed offsets are added to the tangent elevation.

Figure 4-5-A illustrates a crest vertical curve; however, the formulas also are applicable to a sag vertical curve. It is important to recognize the plus or minus characteristics of the grades when computing algebraic differences and whether the offset distances should be added to or subtracted from the tangent elevations.

4-6.03 Recording Plan Grades

The established plan grade elevations should be recorded to the nearest 0.01 foot on the cross-section sheets and on the plan-profile sheets. The plan grade elevations should be displayed along the bottom of the plan-profile sheets in 50-foot increments. In addition, the profile view should show the following:

- the stationing and elevation at each VPC, VPI, and VPT
- the gradients between VPIs
- the length and K-value of each vertical curve

4-7.0 SPECIAL CONSIDERATIONS

4-7.01 Field Recommendations

Additional considerations may be necessary for items such as high-water levels, ditch flow lines, unusual soil conditions, intersections, adjacent property development, or other unusual conditions that may control plan grade elevations.

The plan grade should be designed to ensure adequate elevation above high-water levels for the selected design storm frequency. Freeboard clearance above the high-water level for the design storm frequency should be a minimum four feet for interstate roadways and three feet for all other roadways. In addition, the design storm frequency for the through lanes of an interstate roadway is 50 years.

4-7.02 Ties with Adjoining Projects

A smooth transition should be provided between adjoining projects. Ties should desirably be located whereby no significant grade adjustments are needed beyond the new construction or reconstruction project limits to ensure adequate sight distance. Where existing grades on approaches to a project must be significantly adjusted (more than a few overlay lifts) beyond the designated project limits, grade lines should be properly established throughout the entire transitional areas to ensure adequate sight distance. Alignments throughout the transitions should be compatible with the design speed of the new project and can be used when an adjoining road section is reconstructed. Designs for transitions should also be located such that intersections and median crossovers are not located within or within the close vicinity of the transitions.

See Section 10-5.07 for information on temporary connections for adjoining projects with divided roadways.

4-7.03 Balancing Earthwork

The plan grade should desirably be designed to achieve a balance of earthwork on the project by minimizing the amount of borrow and excess material to the extent feasible. However, design criteria may preclude this objective from being achieved. Examples of such criteria include, but are not limited to:

1. minimum finished grades for hydraulic bridges
2. minimum clearances for overpass bridges
3. allowable plan grade differential between existing lanes and proposed parallel lanes
4. maintaining existing grades near at-grade intersections to facilitate maintenance of traffic and construction

4-7.04 Drainage Structures

Where special controls that impact drainage flow lines exist, the plan grade should meet or exceed the minimum cover over drainage structures. Criteria for cover are presented in the *Standard Drawings*.

4-7.05 Major Structures

The design of plan grades should be carefully coordinated with the bridge design and with the vertical clearance and high-water controls. The designer should adhere to the vertical clearances presented in the geometric design criteria tables in Chapters 2, 12, and 14.

At the Field Inspection and Office Review stages, the designer should verify that the design clearance is provided and that the grade controls shown on the bridge plans and on the roadway plans are identical.

At intersections, the profile should be checked to ensure that adequate ISD is available. See Section 6-6.0 for information on ISD.

4-7.06 Railroad Grade Crossings

The finished surface grade line should match the top of the rails at railroad grade crossings. Minor warping of the surface cross slopes may be warranted.

4-7.07 Adjacent Multilane Roadways

A common plan grade line typically is used for both roadways of divided facilities where the median width is 64 feet or less.

For median widths in excess of 64 feet, independent plan grade lines may be desirable because of the natural terrain. An appreciable grade differential should be avoided between the roadways at median crossovers. Traffic entering from a crossroad may make a wrong-way movement if the pavement of the far roadway is obscured because of a grade differential. Typical safety slopes used in the median may place further restrictions on the grade differential.

A grade differential can be further complicated when a divided facility is provided by using the existing two lanes for one direction of travel and constructing two new lanes for the other direction. For safety, both roadways at crossroads and crossovers should desirably have the same elevation. However, in many instances, the existing two lanes may have less than desirable vertical alignment, and attempts to match the elevations could sacrifice the desirable vertical alignment of the two new lanes. The location of crossovers should be adjusted or eliminated to improve the safety characteristics.

The grade of crossover should desirably not exceed 6%. Even without a crossover, large differences in elevation can make it less feasible to provide proper safety slopes, median drainage, and superelevation development. Table 4-7-A presents the maximum elevation differential between adjacent roadways for various horizontal distances between centerlines.

Plan grades between adjacent roadways at or near the maximum differential should be avoided where feasible. Consideration should be given to reconstructing portions of the existing roadway at these locations.

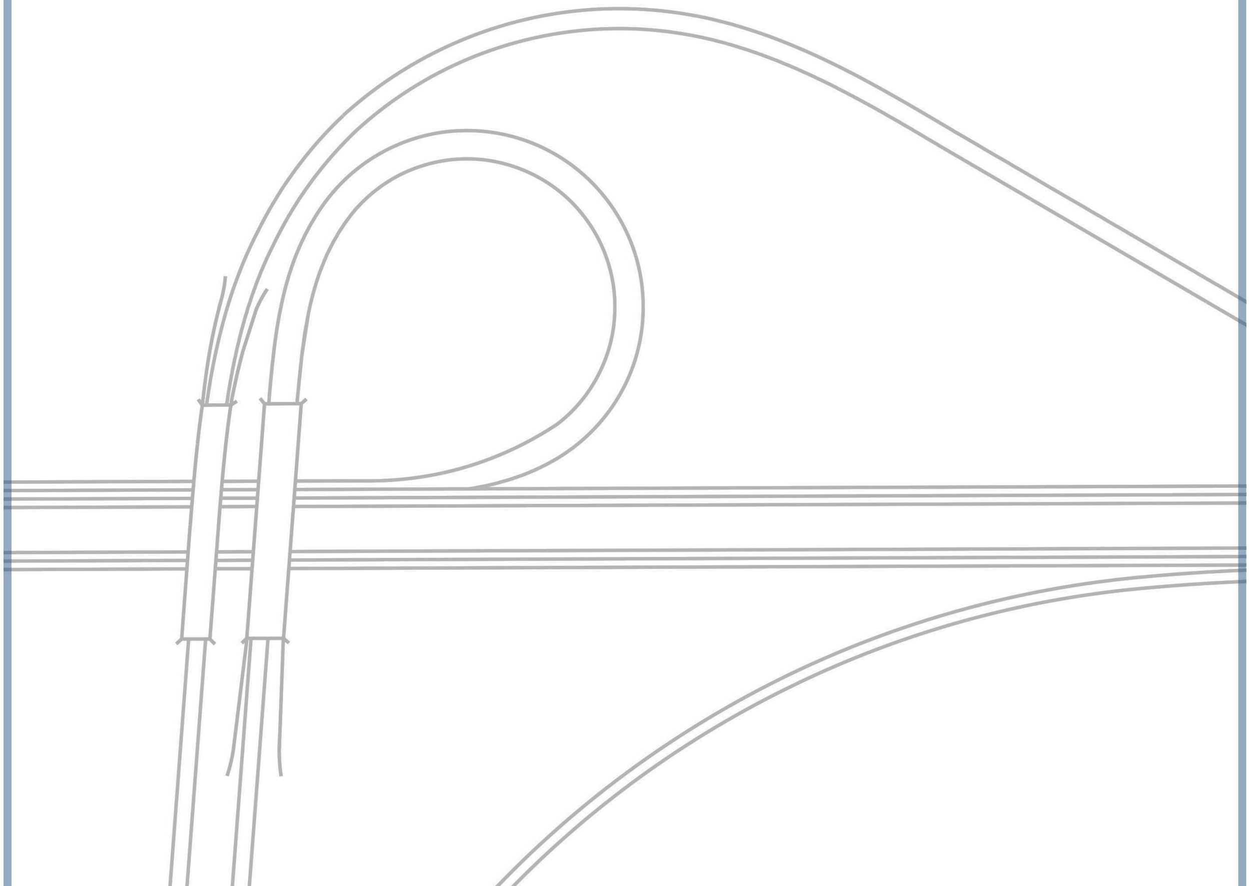
Table 4-7-A
ELEVATION DIFFERENTIAL BETWEEN ADJACENT ROADWAYS

4-Lane Roadway		Maximum Plan Grade Differential (ft)	
Horizontal ℄ to ℄ (ft) Distance	Median Width (ft)	With Crossover	Without Crossover
64	40	1.0	1.0
88	64	2.0	2.0
125	101	4.2	11.3
150	126	5.7	17.5
200	176	8.7	30.0
250	226	11.7	42.5

4-8.0 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2018.

2. *Highway Capacity Manual 2010*, Transportation Research Board, 2010.



CHAPTER 5

3D Models and Earthwork

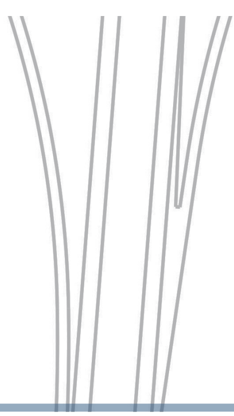


Table of Contents

Chapter 5
3D MODELS AND EARTHWORK

In combination with the horizontal alignment and vertical alignment, cross sections form the third dimension of the roadway plans. Existing ground cross sections should be derived from Digital Terrain Models (DTM) of the existing ground as directed by the *Survey Manual*. For most projects, the Department develops 3D models in its design process and delivers these 3D models upon request to contractors and field personnel for use in Automated Machine Grading (AMG) and for construction inspection.

The scope of the project will dictate whether 3D models or proposed cross sections should be used to process the proposed roadway embankment. For assistance in developing 3D models, the current 3D training manual located on the website should be referenced.

5-1.0 PLOTTING THE ROADWAY TEMPLATE

The roadway template is a graphical representation of the proposed roadway typical section superimposed on the existing ground terrain model. The roadway template establishes the basis for computing the earthwork quantities.

5-1.01 Typical Section Controls

The proposed typical sections for a project define the basic geometric criteria of the roadway template and include:

1. cross slopes
2. safety slopes
3. cut and fill slopes
4. location of ditch line
5. vertical distance from plan grade to subgrade (structure thickness)

5-1.02 Elevation Controls

The elevation of each roadway template should be controlled by the computed plan grade elevation for the corresponding station. When 2-lane roadways and symmetrical, divided multilane roadways have common plan grades, the designer should record the elevation above the point representing the plan grade of the template. When multilane, divided roadways have separate plan grades, the elevation above the plan grade point of each roadway template should be recorded. The accuracy of the grade elevation should be to the nearest 0.01 foot.

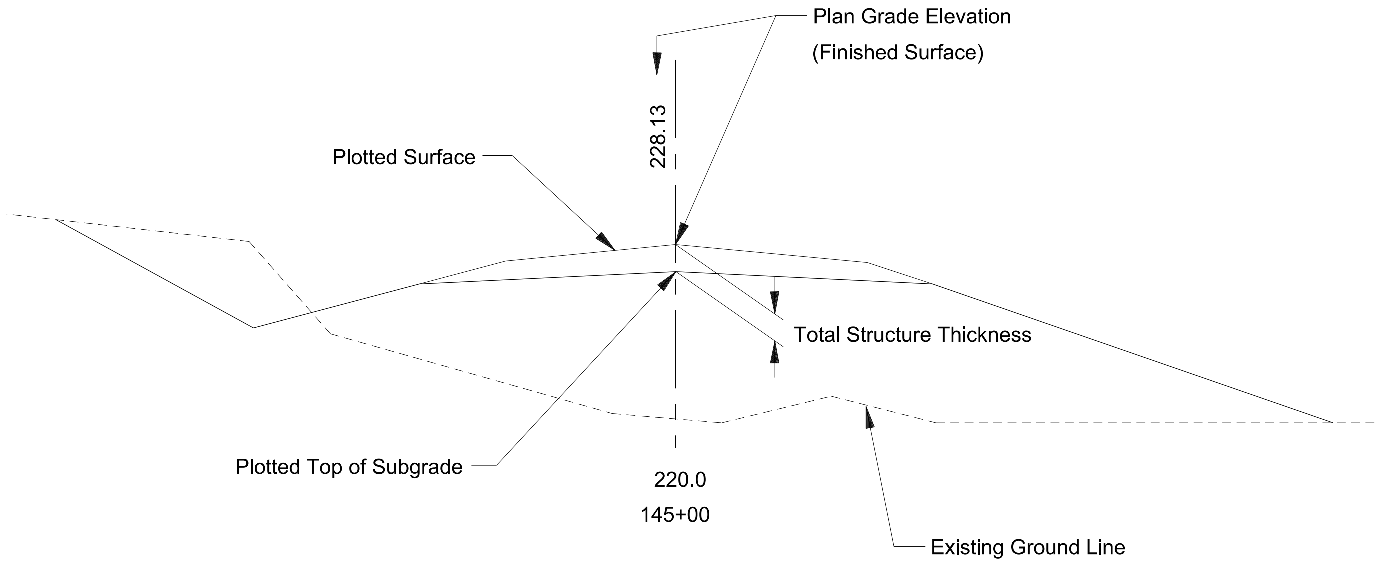
5-1.03 Plotting

A typical plotted roadway template is illustrated in Figure 5-1-A. The principal objective in plotting the roadway template is to define the shape and limits of roadway excavation and embankment sections relative to the existing ground line. Plotting the surfacing elements of the template is necessary to clearly illustrate the lane and shoulder widths, superelevation, intersections, ramps, etc. In urban areas, the roadway surface should be plotted to define relationships between the curbs and the adjacent ground lines and to define drop inlets and other drainage structures.

5-1.04 Items to Show on Cross Sections

5-1.04.01 Roadway Template

In addition to the roadway template, all phased construction elements should be shown (e.g., detour roads, drainage structures, temporary slopes, temporary connections, ditch flow line elevations).



TYPICAL PLOTTED ROADWAY TEMPLATE
Figure 5-1-A

5-1.04.02 Driveways

All driveways should be shown on the cross sections to verify proper connectivity between realigned or regraded driveways with existing driveways within the proposed right of way. Temporary easements should be established for instances when this condition cannot be met. For more information on the recommended design criteria for driveways, see Section 6-9.0 and Section 14-2.07.02 for rural and urban driveways, respectively.

5-1.04.03 Drainage Structures

All drainage structures should be plotted to scale on the appropriate cross sections. These drawings will assist the designer in establishing flow lines and estimated lengths of structures, determining if the proposed grade provides the minimum cover for the structure, heights of inlets, etc.

5-1.04.04 Bench Sections

Where deep cuts or high fills are encountered on a project, the Roadway Design Division should coordinate with the Geotechnical Branch of the Materials Division for their investigation and recommendations. The depth of cut is measured vertically from the subgrade to the top of the backslope. The height of fill is measured vertically from the subgrade to the toe of the slope. Deep cuts and high fills are defined as follows:

- Districts 1, 5, and 6 - cut sections greater than 30 feet and fill sections greater than 40 feet

- Districts 2, 3, and 7 - cut sections greater than 25 feet and fill sections greater than 35 feet

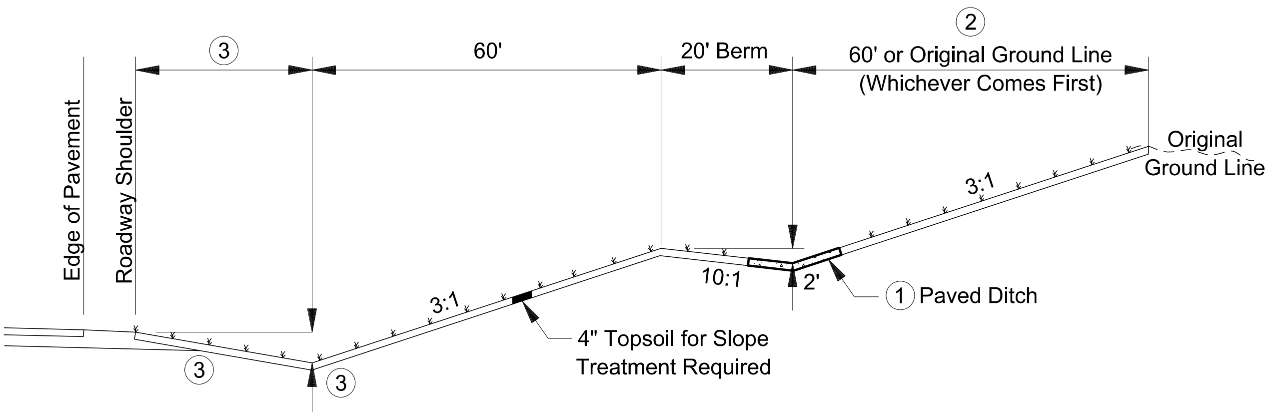
Wherever the above limits are reached, the designer should plot bench sections on the cross sections through the entire cut or fill section before sending the plans to the Geotechnical Branch. Typical bench sections are shown in Figure 5-1-B. Additional bench sections may be necessary when the single bench section does not tie the existing slope to the existing ground within the distances shown in the figure.

Consideration of bench sections should also be given to high fills in proximity to new bridges. When embankment fill heights are 30 feet or greater at a bridge abutment, Bridge Division and Materials Division must be notified to evaluate the proper course of action. In some instances, a cost comparison should be conducted to determine if the bridge should be lengthened to attain a fill height less than 30 feet at the bridge abutment, or if bench sections should be applied to the approach roadway embankment and spill-through slope under the bridge.

The Geotechnical Branch should be provided with the following plan sheets as early in plan development as possible so that the amount of additional proposed right of way can be determined for stable slopes:

1. title sheet
2. typical bench section(s) with station limits indicated
3. plan-profile sheet(s)
4. cross-section sheets

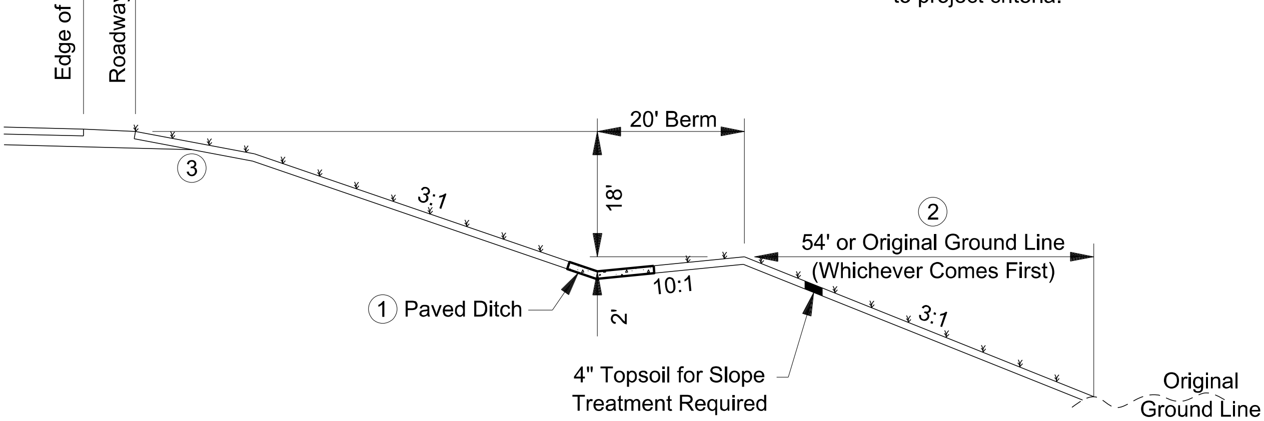
The Geotechnical Branch should confirm the design or provide alternative recommendations for the bench section(s).



Cut Section

Notes:

1. Paved ditch along berms may be necessary as requested by Hydraulics Engineer or applicable District.
2. Bench sections should be repeated until intercepting original ground line.
3. Slope and ditch may vary according to project criteria.



Fill Section

See Section 8.1-0 for guidance on temporary siltation and erosion control measures.

EXAMPLES OF BENCH SECTIONS
Figure 5-1-B

5-2.0 EARTHWORK COMPUTATION

5-2.01 Computer Computations

Earthwork should be computed with GEOPAK or OpenRoads, which also should be used to draw the cross sections and determine quantities using the average end area method. Figure 5-2-A presents the average end area methodology for determining earthwork quantities. The designer should use the design software to produce:

- design soil cut and fill areas of each roadway template
- design soil earthwork volumes

Note: Subgrade volumes should also be calculated by the average end area method from the cross sections.

When the desired earthwork balance has been attained and the final plan grades established, the designer should plot the roadway templates and record the earthwork volumes on the cross-section sheets. Figure 5-2-B illustrates a typical cross-section sheet with recorded computer earthwork volumes. Fill volumes should be placed on the left and cut volumes on the right of the corresponding cross-section sheets.

5-2.02 Shrinkage Factor

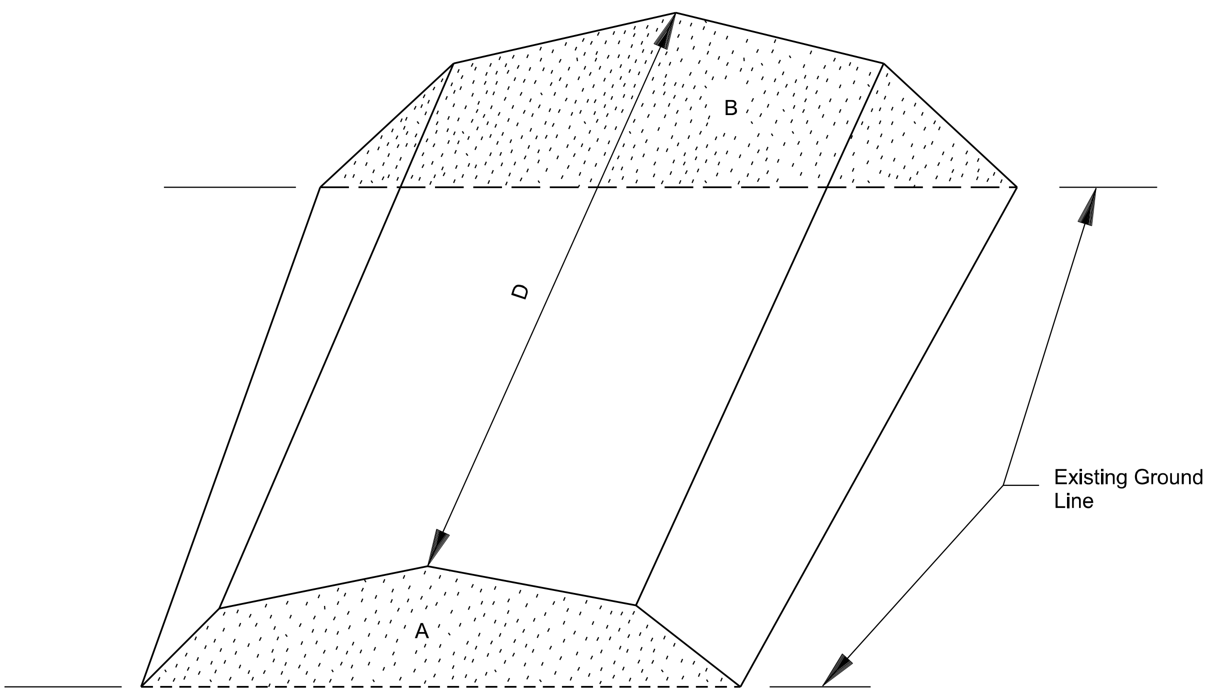
A shrinkage factor should be applied to the earthwork computations. Following are the two methods of applying the shrinkage factor:

- Increase the computed embankment quantity by the shrinkage factor to determine the amount of excavation actually needed to construct the embankments.

- Decrease the computed excavation quantity by the shrinkage factor to determine the quantity of embankment that will be produced.

The first method should be used when a project is to be fully constructed with excavation from within the project site. The second method should be used when the project cannot be fully constructed with on-site material and other borrow excavation must be obtained.

The shrinkage factor will vary with location and soil characteristics.

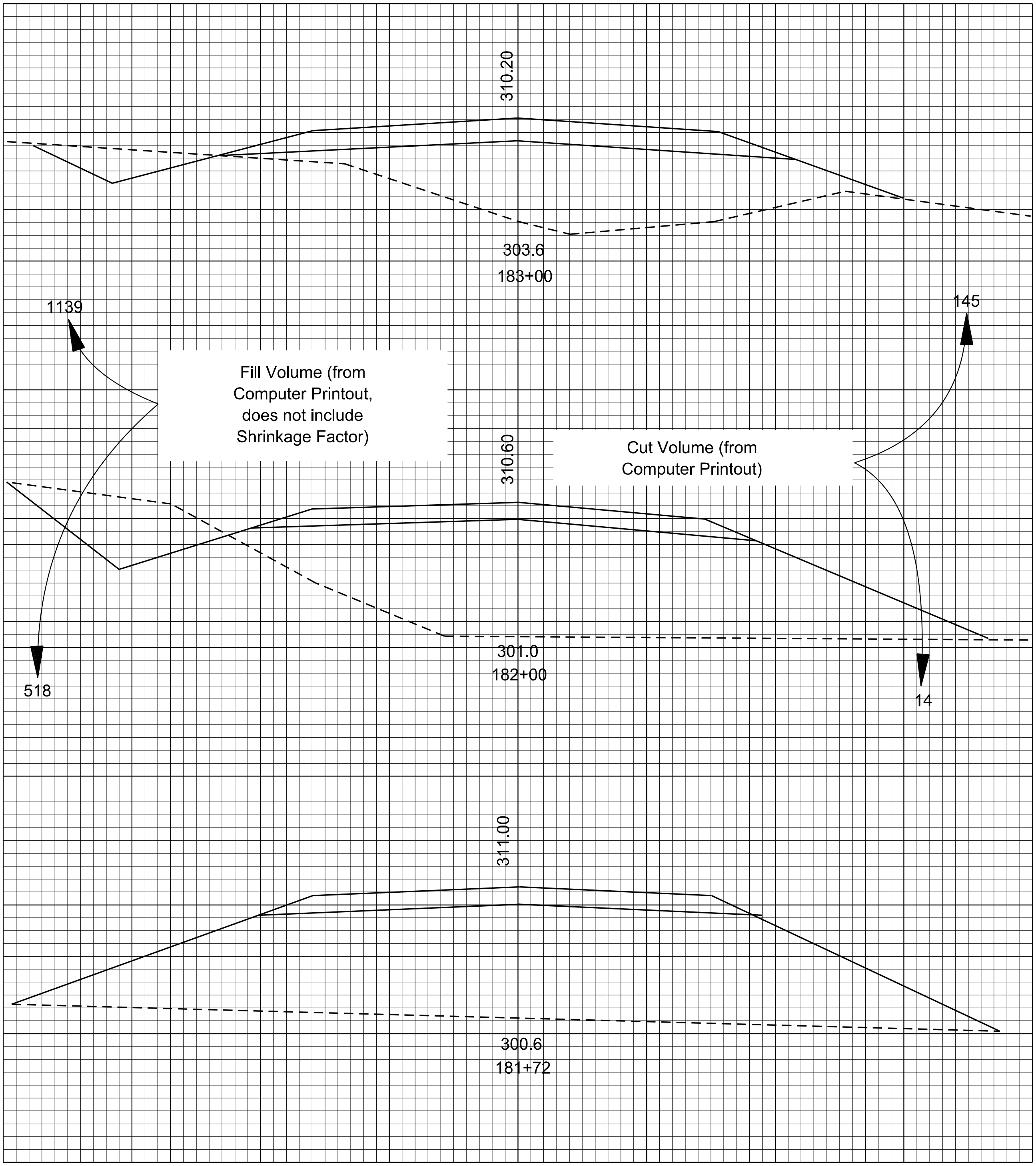


$$V = D\left(\frac{A+B}{2}\right) \div 27 \text{ or } V = \frac{D(A+B)}{54} \qquad \text{(Equation 5-2-1)}$$

Where:

V	=	Volume between sections (cubic yards)
A and B	=	End areas of adjacent sections (square feet)
D	=	Distance between sections (feet)

AVERAGE END AREAS
Figure 5-2-A



TYPICAL CROSS SECTION WITH COMPUTER EARTHWORK VOLUMES
Figure 5-2-B

5-2.03 Earthwork Calculations

Cut and fill lines should be shown on the plan-profile sheets. Any special borrow or excess excavation due to unsuitable material or high-volume-change (HVC) soil should be noted on the cut and fill lines. Sheet totals should include cut, fill, borrow, and/or excess. Special borrow and excess amounts should not be included in the cut and fill totals. The borrow and/or excess that results from cuts and fills should be shown on the estimated quantity sheet. Tables 5-2-A and 5-2-B illustrate example earthwork calculations of excess and borrow conditions, respectively. Table 5-2-C illustrates example earthwork calculations with HVC soil. The last plan-profile sheet should include the cumulative earthwork totals for the entire project.

When less than approximately 15% of the required embankment material is available from excavation within the right of way, the excavation should be identified as Estimated State Furnished Excavation (ESFE) and become the property of the contractor. Table 5-2-D presents an example ESFE calculation. The ESFE should be excavated and used in the embankment, if suitable, and should be paid for as borrow excavation (i.e., Final Measure - Embankment (FME)).

Table 5-2-A
EXAMPLE EARTHWORK CALCULATION
(Unclassified Excavation with Excess Condition)

ESTIMATED EARTHWORK QUANTITIES						
Work Sheet Number	**Cut (yd^3)**	**Fill (yd^3)**	**Borrow Excavation (yd^3)**	**Excess Excavation (yd^3)**	**Unclassified Excavation (yd^3)**	**Remarks**
3	12,321	3414				
4	18,777	4791				
5	7982	2514				
Subtotal	39,080	10,719				
Excess = (Cut) – (Fill)(1+SF)						
= 39,080 – (10,719)(1.25)				25,681		From cuts and fills
Unclassified = (Fill)(1+SF)						
= (10,719 × 1.25)					13,399	From cuts and fills
Project Totals				25,681	13,399	

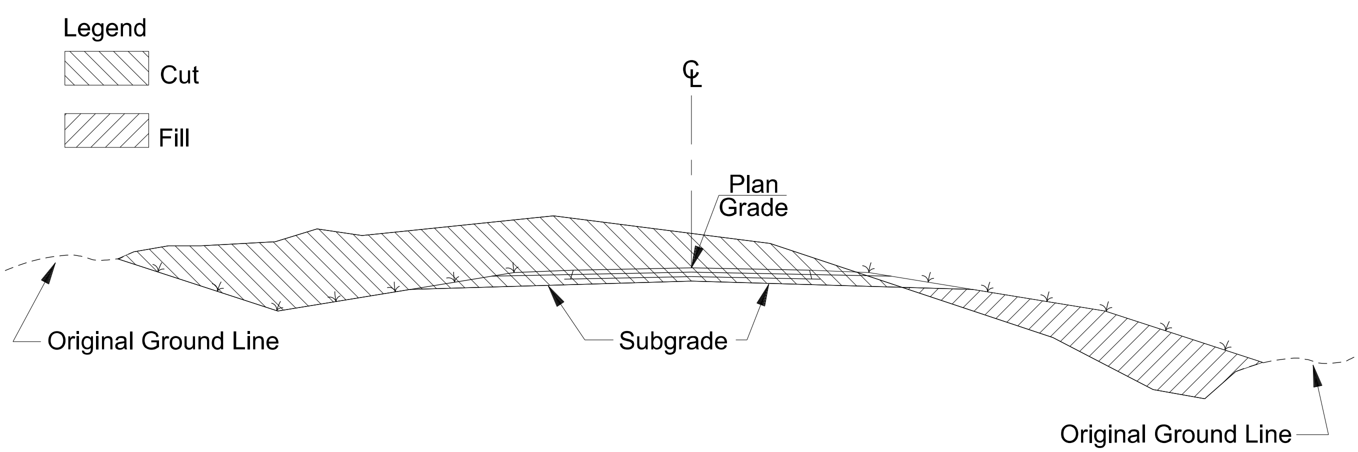


Table 5-2-B
EXAMPLE EARTHWORK CALCULATION
(Unclassified Excavation with Borrow Condition)

ESTIMATED EARTHWORK QUANTITIES						
Work Sheet Number	**Cut (yd^3)**	**Fill (yd^3)**	**Borrow Excavation (yd^3)**	**Excess Excavation (yd^3)**	**Unclassified Excavation (yd^3)**	**Remarks**
3	8514	21,322				
4	10,121	31,716				
5	7850	16,111				
Subtotal	26,485	69,149				
Borrow = Fill – (Cut / (1+SF))						
= 69,149 – (26,485 / 1.25)			47,961			From cuts and fills
Unclassified = Cut					26,485	From cuts and fills
Project Totals			47,961		26,485	

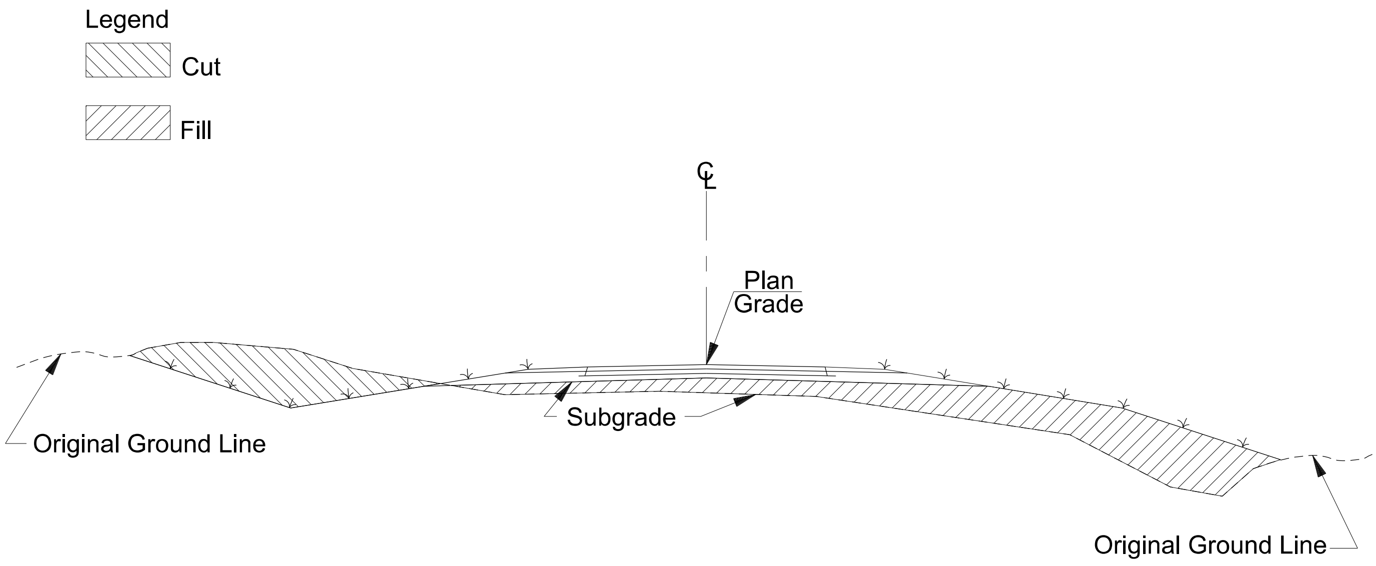


Table 5-2-C
EXAMPLE EARTHWORK CALCULATION
(Unclassified Excavation with HVC and Borrow Condition)

ESTIMATED EARTHWORK QUANTITIES						
Work Sheet Number	**Cut (yd^3)**	**Fill (yd^3)**	**Borrow Excavation (yd^3)**	**Excess Excavation (yd^3)**	**Unclassified Excavation (yd^3)**	**Remarks**
3	24,641	54,923				
4	15,831	7890				
5	2372	741	12,940①	76,360①		
Subtotal	42,844	63,554	12,940①	76,360①		
Unclassified = Cuts					42,844	From cuts and fills
Borrow = Fill – (Cut / (1+SF))						
= 63,554 – (42,844/1.25)			29,279			From cuts and fills
Project Totals			42,219	76,360①	42,844	

Notes:

① *Due to the removal and replacement of high-volume-change (HVC) soil*

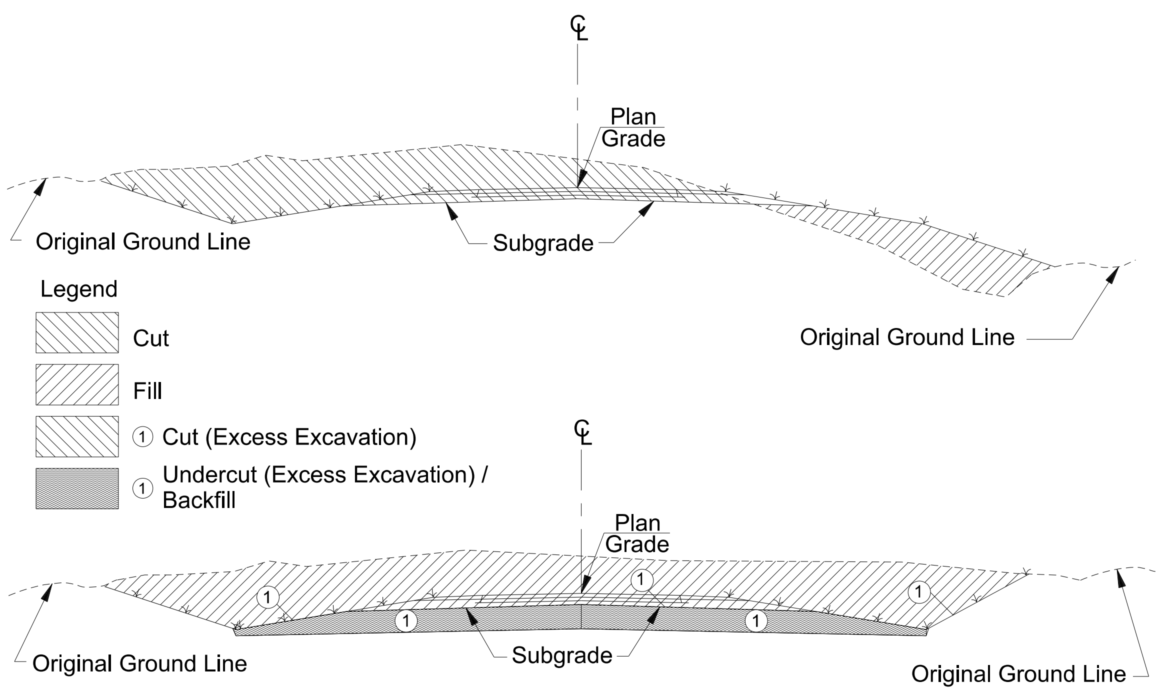
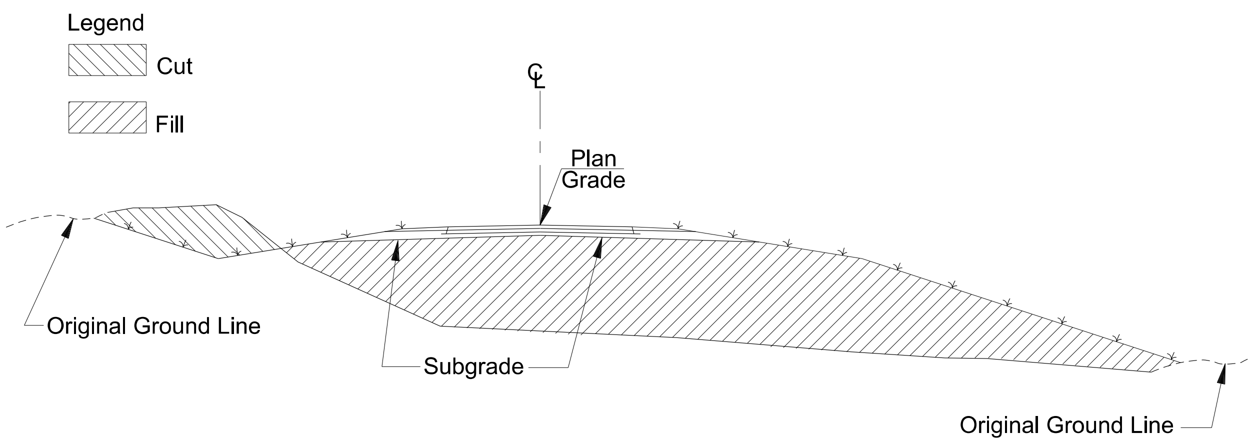


Table 5-2-D
EXAMPLE EARTHWORK CALCULATION
(Estimated State Furnished Excavation Condition)

ESTIMATED EARTHWORK QUANTITIES						
Work Sheet Number	**Cut (yd^3)**	**Fill (yd^3)**	**Borrow Excavation (yd^3)**	**Excess Excavation (yd^3)**	**Unclassified Excavation (yd^3)**	**Remarks**
3	1120	23,770				
4	1325	22,310				
5	1160	28,700				
Subtotal	3605	74,780				From cuts and fills
Borrow = Fill			74,780			From cuts and fills
Project Totals			74,780			

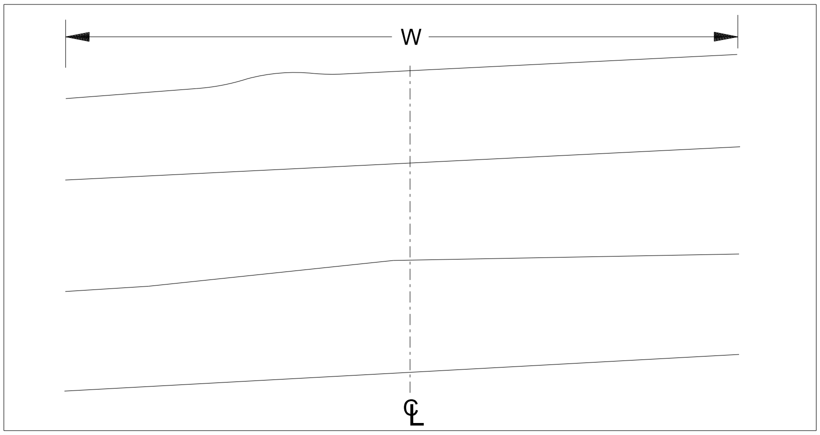
Note: Cut total is less than 15% (+/–) of the total fill and becomes the property of the contractor.



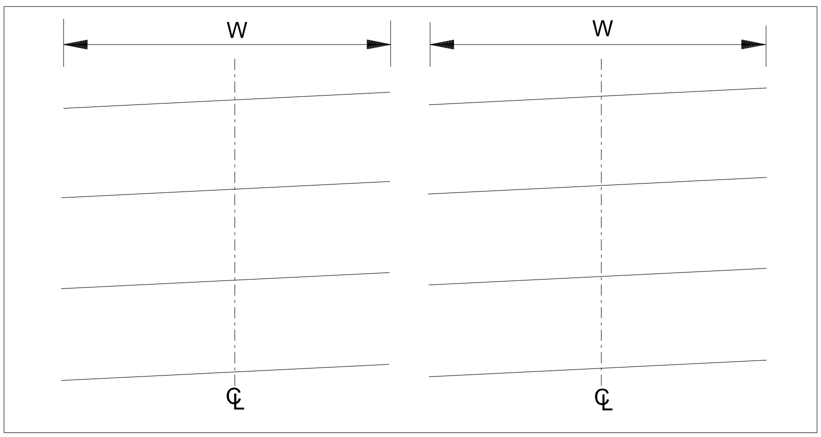
5-3.0 CROSS-SECTION SHEETS

5-3.01 Sheet Layout

The layout of a cross-section sheet depends on the width of cross sections to be plotted and the horizontal scale selected. Figure 5-3-A illustrates various options for cross-section sheet layouts and the total width of cross section (W). These layout options provide the best utilization of space on a sheet, keeping the total number of cross-section sheets to a minimum.



Option 1 : W ≥ 200 ft



Option 2 : W ≤ 150 ft

CROSS-SECTION SHEET LAYOUT
Figure 5-3-A

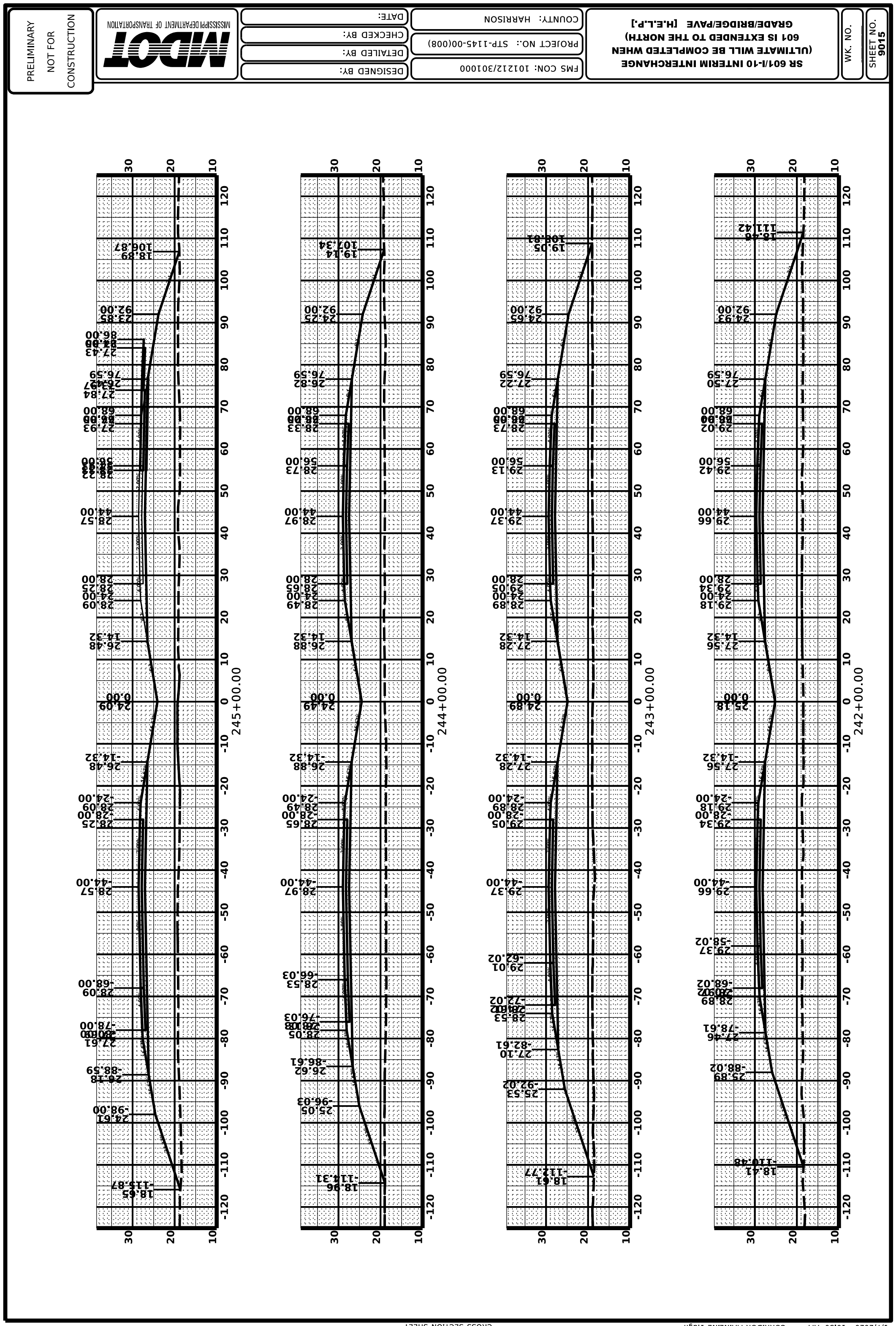
5-3.02 Sheet Identification

Sequential sheet numbers, beginning with 9001, should be assigned as the cross-section sheets are plotted. The mainline cross sections should be sequentially plotted from beginning to end without interruption. Any supplemental cross sections, including interchange ramps, local roads, channel changes, etc., should be grouped at the end of the mainline cross sections with each sheet clearly labeled in the title block. The designer should designate any cross section that is skewed (e.g., along existing pipe) as a skewed cross section.

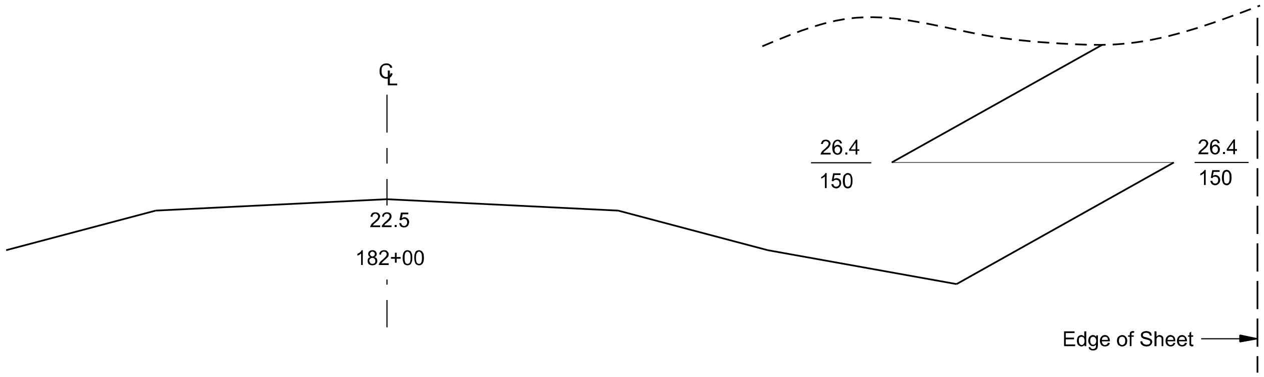
5-3.03 Plotting

Figure 5-3-B illustrates a typical cross-section sheet with plotted sections of the existing ground line. The cross-sections should be plotted according to the following information:

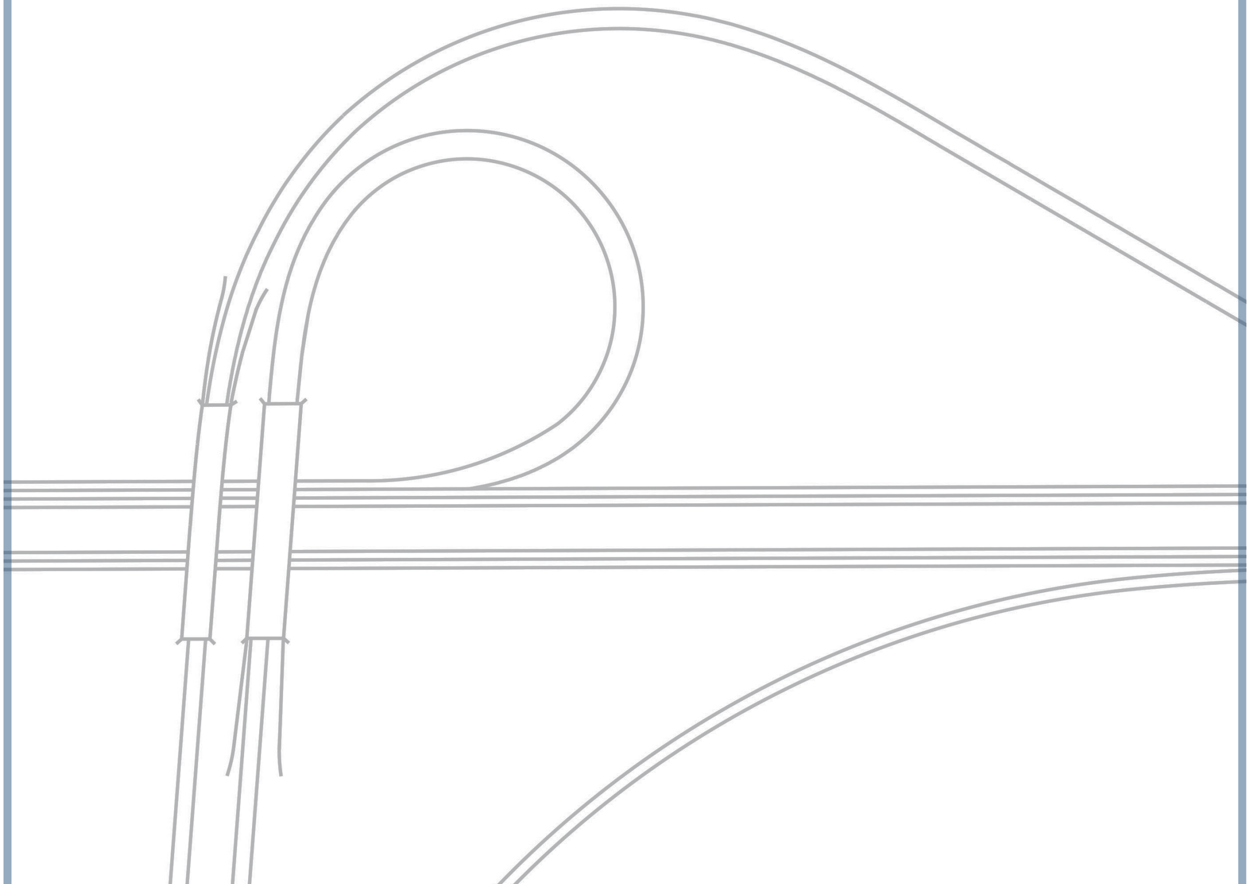
1. A zero (0) should be placed on the bottom border of each sheet to identify the vertical line selected for the centerline of the cross sections.
2. Horizontal distances to the left and to the right of the centerline should be provided on the bottom border of the sheet at 10-foot intervals.
3. The accuracy required for the existing ground elevation at the centerline should be to the nearest 0.1 foot (0.01 foot in the case of existing pavement) and recorded immediately below the plotted cross section. The stationing of each cross section should be recorded directly below the centerline elevation.
4. The vertical spacing between plotted cross sections should be sufficient to avoid overlapping.
5. Where an occasional cross section extends horizontally beyond the edge of the sheet, the slope line may be continued by an offset match line as illustrated in Figure 5-3-C.



SAMPLE CROSS-SECTION SHEET
Figure 5-3-B

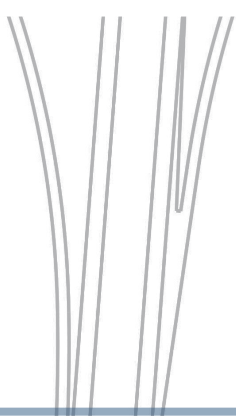


CROSS-SECTION OFFSET MATCH LINE
Figure 5-3-C



CHAPTER 6

At-Grade Intersections



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Chapter 6
AT-GRADE INTERSECTIONS

Intersections are important elements of the roadway system, and the operational efficiency, capacity, safety, and cost of the system depend largely upon their design. The main objective of intersection design is to facilitate the safe and efficient movement of passenger cars, buses, trucks, pedestrians, and bicycles traversing the intersection.

This chapter discusses the geometric design criteria for at-grade intersections, which apply to intersections on new construction/reconstruction projects and projects for improvements to existing roadways. These criteria also apply to intersections on both urban and rural roadways.

6-1.0 GENERAL DESIGN CONTROLS

6-1.01 General Design Considerations

Several elements should be balanced to produce a safe and efficient intersection design. Basic elements that should be considered are:

1. Human Factors

   a. driving habits
   b. driver expectancy
   c. decision and reaction time
   d. conformance to natural paths of movement
   e. pedestrian use and habits

2. Traffic Considerations

   a. capacity
   b. ADT and DHV
   c. vehicular composition
   d. turning movements
   e. vehicle speeds
   f. safety

3. Physical Elements

   a. character and line of abutting property
   b. topography
   c. right of way
   d. horizontal alignment
   e. vertical alignment
   f. coordination of vertical profiles of the intersecting roads
   g. coordination of horizontal and vertical alignment for intersections on curves
   h. available sight distance

i. intersection angle
j. conflict area
k. geometrics
l. channelization
m. traffic control devices
n. lighting
o. safety features
p. bicycle traffic
q. environmental impact
r. drainage

4. Economic Factors

   a. cost of improvements
   b. crash history
   c. effects on adjacent property (e.g., access to businesses)
   d. impact on energy consumption

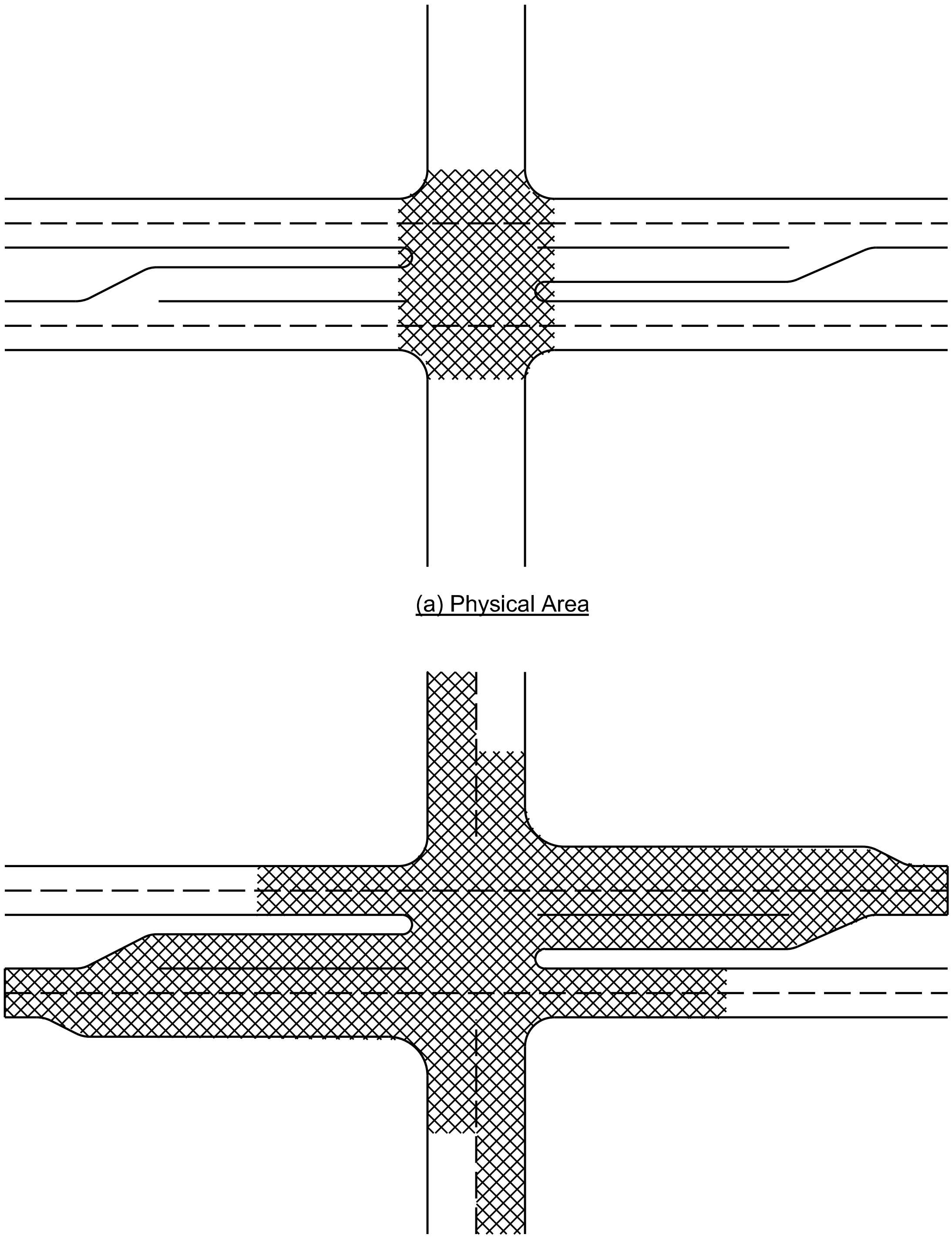
5. Functional Intersection Area – An intersection can be defined by both functional and physical areas as illustrated in Figure 6-1-A. The functional area of the intersection (including any auxiliary lanes and their associated channelization) extends both upstream and downstream from the physical intersection area. Closely-spaced adjacent intersections or frontage roads may also be included in the functional area. The essence of good intersection design dictates that the physical elements be designed to minimize the potential conflicts among cars, trucks, buses, bicycles, and pedestrians. In addition, human factors of drivers and pedestrians should be taken into account while keeping costs and impacts to a minimum.

6-1.02 Design Vehicles

At intersections, the skew angle, turning radii, and widths of turning roadways should be designed to accommodate the characteristics of vehicular operation. The selection of a design vehicle should typically be based on the largest vehicle that is expected to use the intersection. However, adequate radii for vehicle operation should be balanced against the needs of pedestrians and the presence of right of way and/or environmental constraints. The basic design vehicles used for intersection design are:

1. P — passenger car and both light panel and pickup trucks
2. SU-40 — single-unit truck (three axles)
3. S-BUS-40 — large school bus
4. WB-62 — semitrailer combination with an overall wheelbase of 62 feet
5. WB-67 — semitrailer combination with an overall wheelbase of 67 feet

The S-BUS-40 is generally the smallest vehicle used in the design of intersections. Even in residential areas, school buses negotiate turns with some frequency. One of the semitrailer combinations should be used for design on roadways with regular truck traffic. Section 6-2.01 discusses the selection of a vehicle specifically for turning radii designs.



(a) Physical Area

(b) Functional Intersection Area

PHYSICAL AND FUNCTIONAL INTERSECTION AREAS
Figure 6-1-A

6-1.03 Horizontal Alignment

Roadways should desirably intersect on tangent sections and at right angles, except for intersections where roundabouts are utilized, but several factors should be considered when designing an intersection. Conditions and constraints may vary at each site; therefore, sound engineering judgment should be used in order to provide the most feasible design for each situation. The following subsections provide guidelines and criteria for designing new/reconstructed intersections and improvements to existing intersections.

6-1.03.01 Design Speed

The design speed of the crossing route should typically be based on its functional classification and the amount of traffic. However, operating speeds below the typical design speed are expected as vehicles on the crossing route approach an intersection. Therefore, a lower design speed may be more appropriate and should be considered.

While a lower design speed should be considered for a crossing route as it approaches an intersection, short-radius horizontal curves should desirably not be used to achieve right-angle intersections on crossing route approaches. Such design restricts sight distance and potentially introduces opposing lane encroachments on crossing route approaches.

6-1.03.02 Angle of Intersection

Crossing routes should desirably intersect the mainline at or as close to 90 degrees as feasible. Roadways intersecting at acute angles are undesirable for the following reasons:

- Extensive turning roadway areas are needed.
- The exposure time to vehicles and pedestrians crossing the main traffic flow is increased.
- Driver visibility from the crossing route may be limited.
- Vehicles, especially trucks, turning from the mainline may have difficulty turning onto the crossing route without exceeding the acceptable amount of encroachment (as shown in Table 6-2-D).

Some deviation is permissible, though, when constraints exist that prohibit a 90-degree intersection. In these cases, the angle of intersection should be no less than 75 degrees (greater than 15-degree skew).

6-1.03.03 Superelevation

Superelevation may complicate intersection design, and horizontal curvature may contribute to restricted sight distance. A lower design speed for the crossing route near the intersection allows horizontal curves to have a lower superelevation rate, resulting in a more reasonable transition from the superelevation of the crossing route to the vertical alignment of the mainline. The designer should check to ensure that the tangent distance between the intersection and the closest horizontal curve on the crossing route is long enough to allow for a reasonable superelevation transition.

6-1.03.04 Existing Intersections

The following guidelines and criteria should apply only to existing intersections within the limits of a project, depending on the scope of the project:

1. Existing skewed intersections with an angle of intersection less than 90 degrees (0-degree skew) but greater than or equal to 75 degrees (less than 15-degree skew) are typically acceptable to remain in place without realignment of the crossing route. However, if existing conditions and crash patterns indicate the need for realignment of the crossing route as a feasible mitigation measure, such realignment design should provide for a non-skewed intersection, if possible. Figure 6-1-B illustrates an existing 3-legged intersection that is acceptable to remain in place, but also illustrates a desirable realignment to a 90-degree intersection if a realignment is considered.

2. Existing skewed intersections with an angle of intersection less than 75 degrees (greater than 15-degree skew) should typically be realigned to 90 degrees or as close to 90 degrees as feasible, but should be no less than 75 degrees. Figure 6-1-C shows a realignment for a 3-legged intersection.

3. Existing skewed 4-legged intersections with angles of intersection less than 75 degrees (greater than 15-degree skew) should be realigned to create a new 4-legged intersection that intersects as close to 90 degrees as feasible. Listed below are realignment options that should be considered:

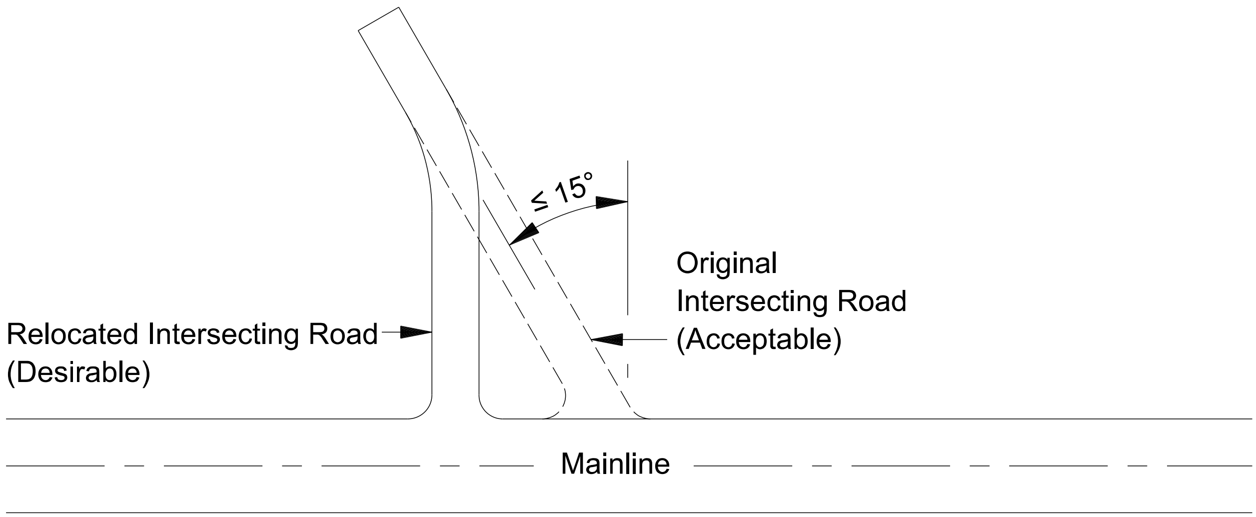
    a. Desirably, each of the skewed crossing routes should be realigned an approximately equal amount in order to minimize the impacts on each side of the mainline, which should also result in better horizontal alignments for each crossing route. See Figure 6-1-D.

    b. If conditions exist that prohibit this type of realignment for both crossing routes, one crossing route may be realigned significantly more than the other as shown in Figure 6-1-E(a).

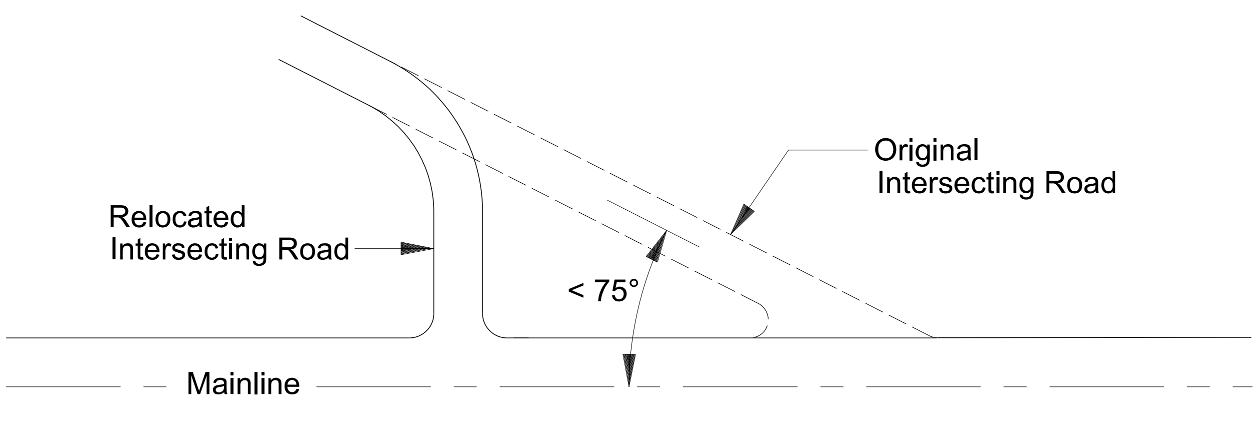
    c. Another acceptable, but less desirable, realignment option is to realign each crossing route such that separate offset 3-legged intersections are created as shown in Figure 6-1-E(b). This option should only be considered for crossing routes that are functionally classified as local roadways. If the offset intersection design is such that the local-roadway vehicles make a right turn onto the mainline and then a left turn onto the next crossing route (as shown in the figure), the mainline may need to be widened between the two intersections to accommodate the left-turning vehicles.

4. Factors including, but not limited to, horizontal alignment, vertical alignment, access control, right of way, and environmental constraints may limit the ability to meet the realignment criteria.

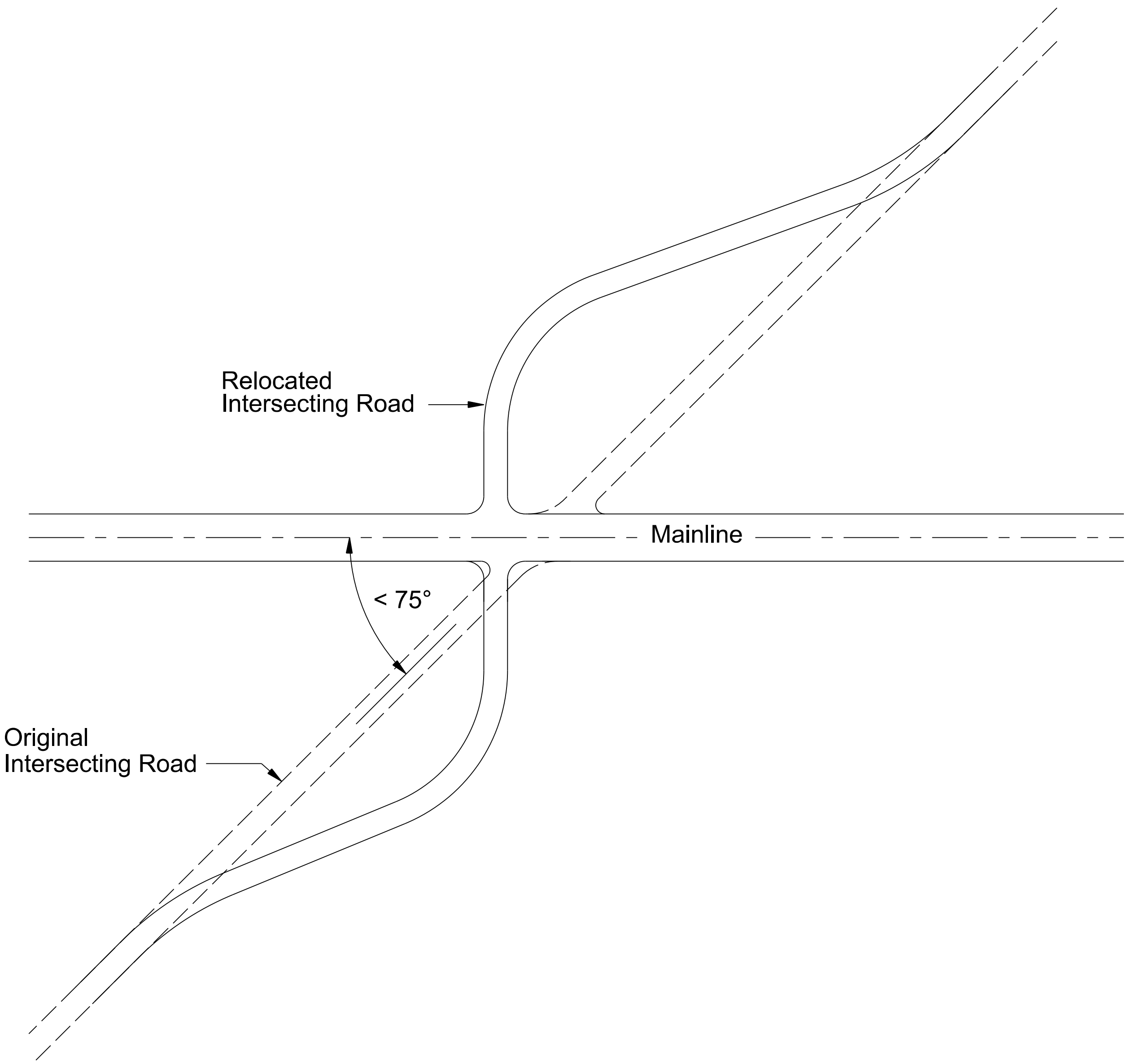
5. Roadways with low traffic and/or little or no crash pattern may not warrant realignment.



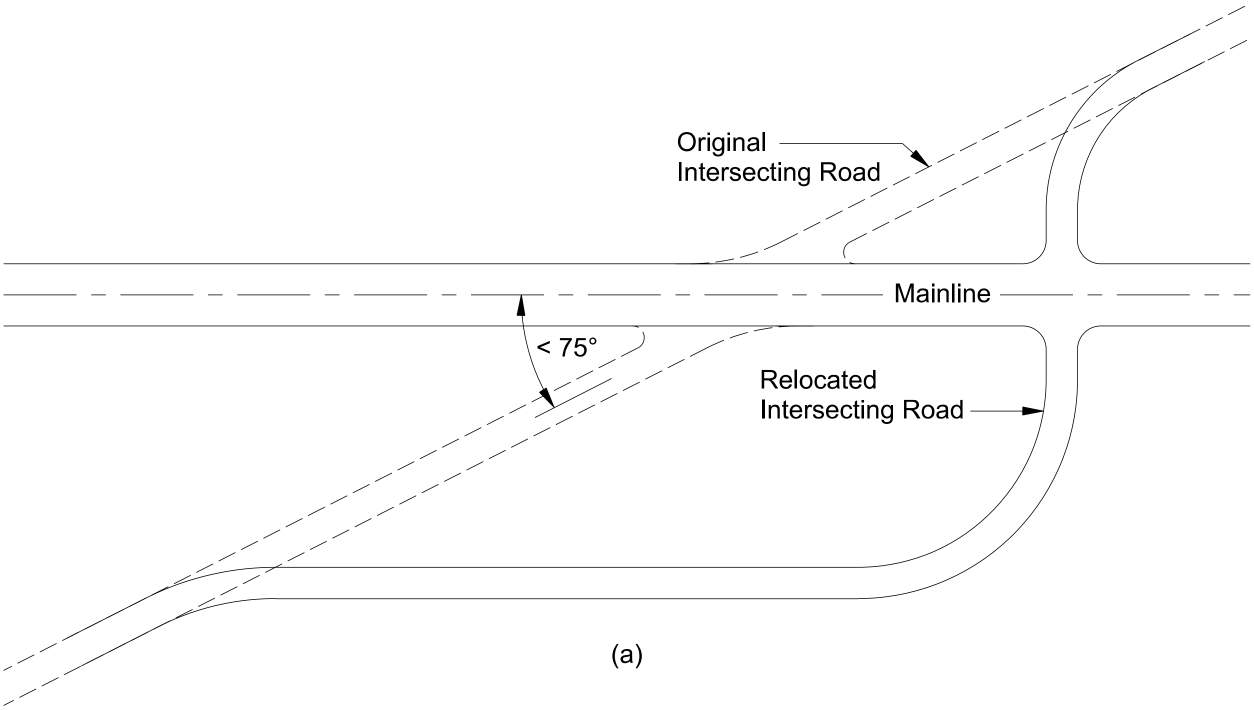
ACCEPTABLE SKEWED EXISTING INTERSECTION
Figure 6-1-B



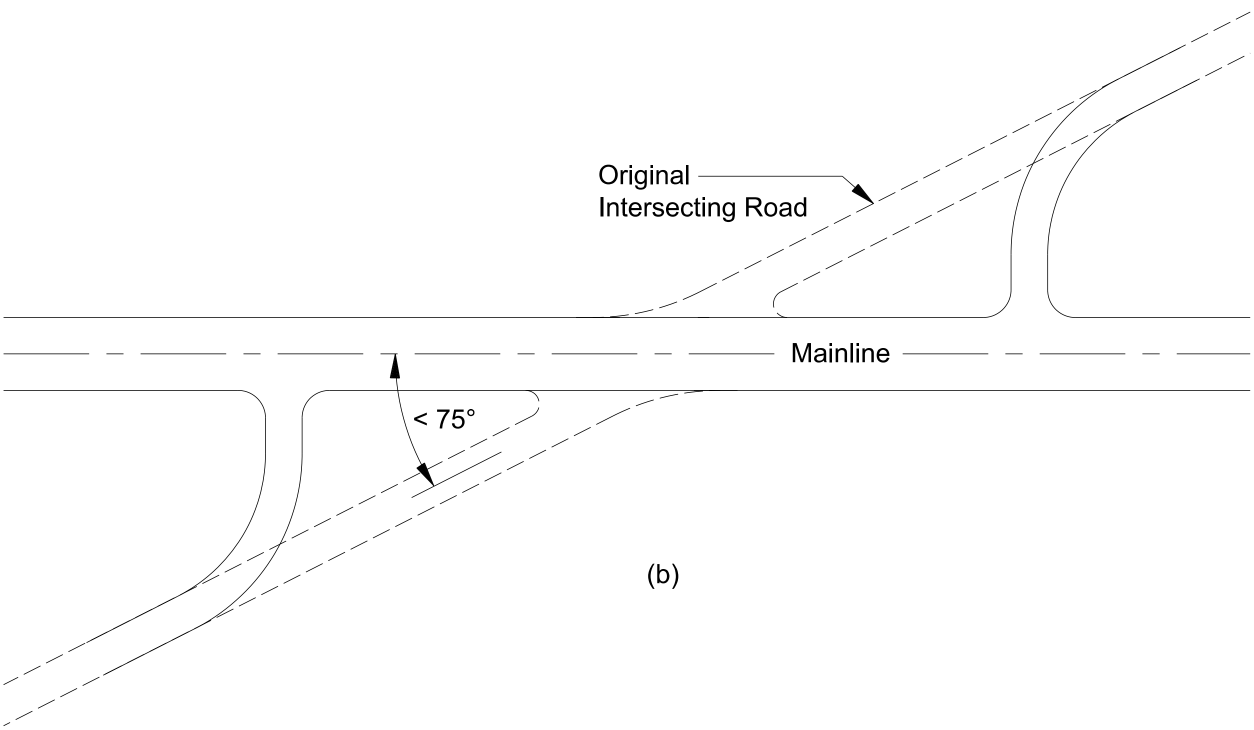
REALIGNMENT FOR SKEWED 3-LEGGED INTERSECTION
Figure 6-1-C



PREFERRED REALIGNMENT FOR SKEWED 4-LEGGED INTERSECTION
Figure 6-1-D



(a)



(b)

OTHER REALIGNMENT OPTIONS FOR SKEWED 4-LEGGED INTERSECTION
Figure 6-1-E

6-1.04 Vertical Alignment

The gradients of crossing routes should desirably be as flat as feasible near the intersection to provide a storage platform for stopped vehicles, but the vertical alignment should still provide adequate surface drainage. Grades steeper than 3% should preferably be avoided near an intersection, as they may result in changes to several design elements to sustain operations equivalent to those on level roads (e.g., Stopping Sight Distance (SSD), Intersection Sight Distance (ISD)). Where it is infeasible to provide grades of 3% or less near an intersection, grades should not exceed 6% and should include a corresponding adjustment in specific design elements.

Intersections with stop control should desirably maintain the vertical alignment and cross section of the mainline through the intersection, with the crossing route being transitioned to fit the vertical alignment of the mainline. At signalized intersections, or at intersections where the roadways are nearly equal in importance, both roadways should desirably be transitioned to a plane section. Where compromises are proposed between the two roadways, the smoother riding characteristics should preferably be provided for the roadway with higher traffic volumes and operating speeds.

6-1.05 Capacity and Level of Service

A capacity analysis should be considered before completing the detailed design of any intersection where the crossroad ADT is 2000 or more. The capacity analysis may influence several geometric design features, including number of approach lanes, lane widths, channelization, and number of departure lanes. The Level of Service (LOS) threshold should be based on the future design year, typically 20 years for new construction/reconstruction and 10 years for 3R construction. Recommendations for LOS are provided in the geometric design criteria tables in Chapters 2, 12, and 14. When the levels of service threshold and design traffic volumes have been determined, the *Highway Capacity Manual* should be used for the capacity analysis.

6-1.06 Types of Intersections

The basic types of at-grade intersections are 3-legged or 4-legged intersections, which may be channelized or non-channelized. See Section 6-7.0 for more information on channelized intersections. Other intersection types that may be used include roundabouts, continuous flow intersections (CFI), and J-turn intersections. Principal factors affecting the selection of intersection type and its design characteristics are traffic volumes, turning movements, character or composition of traffic, design speed, angle of intersection, and right of way or environmental constraints.

6-1.07 Intersection Spacing

Short distances between intersections tend to impede traffic operations and should be avoided where feasible. For example, two intersections that are close together may be considered as one intersection for signalization purposes. To operate safely, each leg of the intersection may warrant a separate green cycle, thereby greatly reducing the capacity for both intersections. To operate efficiently, the minimum spacing for signalized intersections should be 1/3 of a mile in urban areas or 2/3 of a mile in rural areas. See the *Access Management Manual* for more information.

In general, all new non-signalized intersections should be spaced such that no conflicting movements exist within the functional area of each adjacent intersection. If intersections are too close together, drivers tend to encroach into the opposing lanes (corner cutting) so they can make their turning maneuvers in one movement.

6-2.0 TURNING RADIUS DESIGN

Turning radius treatments are key design elements that influence operations, safety, and construction costs of at-grade intersections.

6-2.01 Selection of Design Vehicle

The area type and functional classification of intersecting roadways should be considered when selecting the design vehicle. However, consideration should also be given to balance the needs for vehicle operation against the needs for pedestrian users and the presence of right of way and/or environmental constraints. Types of design vehicles for intersections are discussed in Section 6-1.02. Table 6-2-A and Table 6-2-B present the preferred design vehicle selection according to the crossing route functional classification for rural intersections and urban intersections, respectively.

Interchange ramp terminals, which should be designed in accordance with the criteria presented in this chapter, should typically be designed using a WB-67 design vehicle for arterial crossroads, other intersections on state highways, and industrialized roadways that carry high volumes of traffic or provide local access for large trucks. Engineering judgment should be used when deciding which intersections should be designed according to the turning characteristics of large trucks. Some intersections (e.g., those near truck stops, factories, warehouses, and manufacturing plants) are obvious candidates.

Some intersections may be designed with more than one design vehicle. For example, an intersection may be channelized with a raised island and a turning roadway that will accommodate large trucks, but additional channelization with pavement markings may be included to provide a smaller turning roadway for smaller vehicles. See Section 6-7.0 for more information on channelized intersections and turning roadways.

Table 6-2-A
SELECTION OF DESIGN VEHICLE
(Rural Intersections)

Functional Classification	Design Vehicle
Arterials	WB-67
Collectors	S-BUS-40*
Local Roads	S-BUS-40*

**The need for using the WB-67 as the design vehicle should be evaluated.*

Table 6-2-B
SELECTION OF DESIGN VEHICLE
(Urban Intersections)

Functional Classification	Design Vehicle	
	Desirable	Minimum
Arterials	WB-67	WB-62
Collectors	WB-62	S-BUS-40
Local Roads	S-BUS-40	P

Note: Consideration should also be given to the needs for pedestrian users and the presence of right of way and/or environmental constraints.

6-2.02 Turning Design Criteria

6-2.02.01 Rural Intersections

The following criteria should apply to the crossing route:

- 0 – 300 ADT – The turning radius design should typically be a 65-foot simple curve (based on WB-62 or WB-67 design vehicle) with a taper offset on both the mainline and the crossing route. Recommended radii for design vehicles are shown in Table 6-2-C; however, the radius selected should be based on a balance between vehicular operation and the presence of right of way and/or environmental constraints.

- Greater than 300 ADT – The intersection may warrant channelization. See Section 6-7.0 for information on channelization.

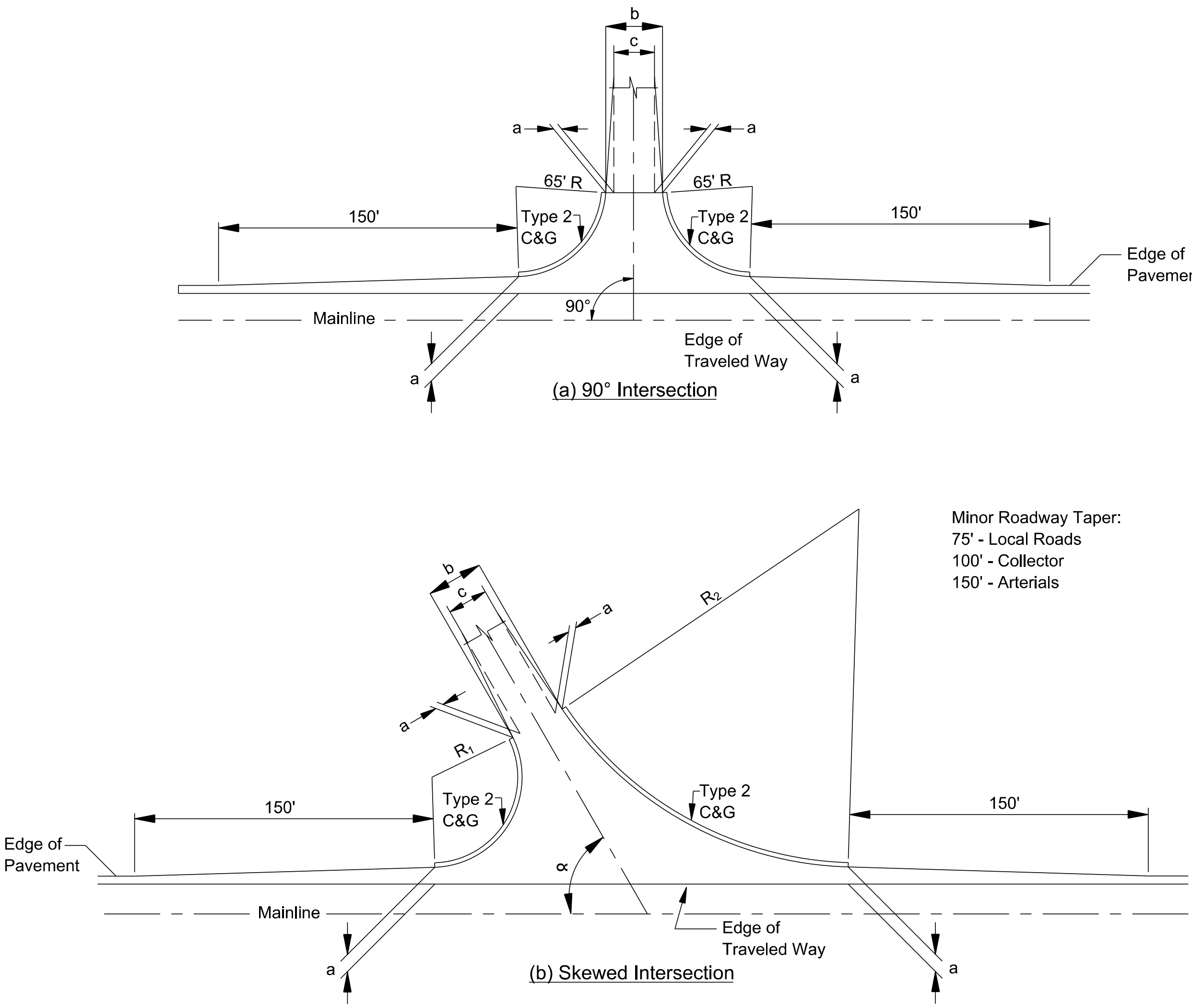
Table 6-2-C
RECOMMENDED CURB RADII
(Non-Channelized Intersections)

Design Vehicle	Radius (ft)
P	20
S-BUS-40	35
SU-40	40
WB-62	65
WB- 67	65

Notes:

1. *Criteria in the table are applicable to intersections where:*
   a. *encroachment onto opposing lanes should be minimal (see Section 6-2.03 for further discussion)*
   b. *the angle of turn is 90° ± 20°*
   c. *parking lanes or shoulders are not available to provide additional width for the turning maneuver (see Section 6-2.05 for further discussion)*
2. *Where a simple radius is considered undesirable for the intersection turn, a compound curve should be considered.*
3. *For radii larger than 65 feet, a turning roadway should be considered. See Section 6-7.0.*
4. *Consideration should also be given to balance the radii for vehicle operation against the needs of pedestrian users and the presence of right of way and/or environmental constraints. See Section 6-2.05 for more information on designing for pedestrians at intersections.*

Figure 6-2-A illustrates a typical design for both a 90-degree and a skewed rural intersection, which include a simple curve radius and tapers on both the mainline and the crossing route. Desirably, the angle of intersection should be 90 degrees, which should allow a relatively easy maneuver for vehicles entering or exiting the mainline. The design should be checked to ensure the turning radius can adequately accommodate the selected design vehicle. See Section 6-1.03 for additional information on horizontal alignment of intersecting roadways.



a = Shoulder width. *Note: Where shoulder width is less than five feet, a minimum 5-foot offset should be provided.*
b = Traveled way plus 10 feet minimum
c = Variable (20 feet minimum)
α = Angle of Intersection (90° desirable, 75° minimum)
R_1 = 65 feet
R_2 = Variable (radius point opposite smaller radius point)

SIMPLE RURAL INTERSECTION WITH TAPER OFFSETS ON BOTH ROADWAYS (Non-Channelized)
Figure 6-2-A

6-2.02.02 Urban Intersections

The same criteria presented in Section 6-2.02.01 for rural intersections should desirably be applied to urban intersections. However, urban intersections typically involve other factors and constraints that limit the feasibility of such a design. The design of an urban intersection should be based upon a balanced approach to accommodate all anticipated modes of traffic, while also taking into consideration any potential right of way and/or environmental impacts.

Figure 6-2-B illustrates a typical design for both a 90-degree and a skewed urban intersection. Although the figure includes tapers only on the mainline, tapers should desirably also be included on the intersecting roadway. However, tapers may not be feasible due to right of way constraints; therefore, a simple curve radius without any tapers may be used. Other details in the figure may vary based on the following considerations:

1. The turning radius may vary based on the selected design vehicle, pedestrian facilities, turning volumes, right of way and/or environmental constraints, and allowable encroachment into opposing lanes of travel. See Section 6-2.03 for guidelines on acceptable encroachment into opposing lanes of travel.

2. For projects on existing urban routes, right of way constraints may prohibit the use of the turning radii shown in Table 6-2-C. In these cases, the following radii may be used:

    a. 15 to 25 feet at minor crossing routes where chances are minimal that trucks will turn

    b. 25 feet where space permits

    c. 30 feet or more for occasional trucks so there will be minimal encroachment into opposing lanes

    d. 40 feet or more where there are frequent turns by large trucks and buses

3. If the crossing route does not include curb and gutter and the shoulder width of the crossing route is less than two feet, the curb and gutter should stop a minimum distance of 10 feet from the radius point along the crossing route (as shown by distance “d” in Figure 6-2-B(a)).

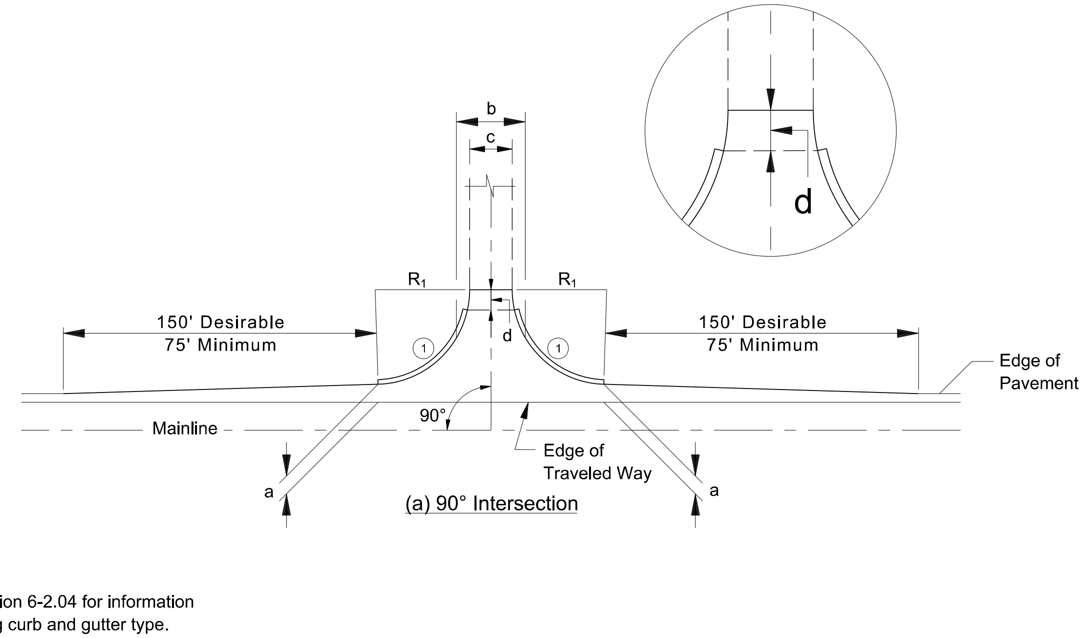
4. If the shoulder width on the crossing route is greater than or equal to two feet, the curb and gutter should extend to the radius point along the crossing route.

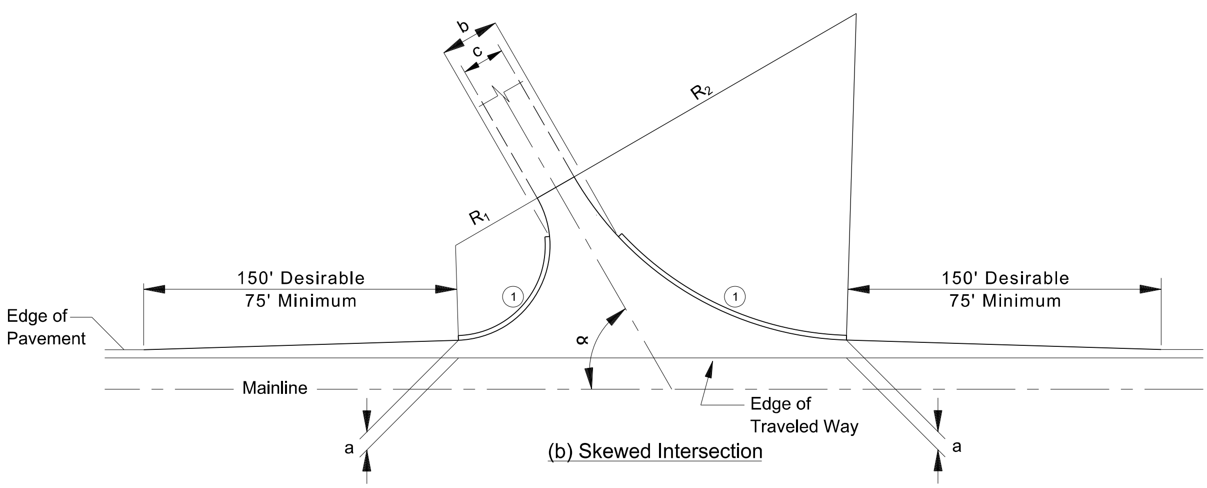
5. If tapers are used along the mainline, the minimum length is 75 feet.

6. See Section 6-2.04 for information on curb and gutter along the turning radii.

Where large vehicles are expected, the overhang of the vehicle as it makes the turn should be considered when determining the location of objects to minimize the probability of the objects being struck by the vehicle. Ideally, objects should be located as far away as feasible, with a desirable lateral offset of six feet from the face of the curb for curbed roadways. However, where right of way or environmental constraints exist, a minimum distance of three feet may be provided.

Where intersections include appurtenances that accommodate pedestrians and bicyclists, the lateral offset should be a minimum of four feet.





a = Shoulder width (Where the shoulder width is less than two feet, a minimum 2-foot offset should be provided.
b = Traveled way plus four feet minimum
c = Variable (20 feet minimum)
d = 10 feet minimum
α = Angle of Intersection (90° desirable, 75° minimum)
R_1 = Variable (based on a balance between the radii needed for vehicle operation against the needs of pedestrian users and the presence of right of way and/or environmental constraints)
R_2 = Variable (radius point opposite smaller radius point)

SIMPLE URBAN INTERSECTION WITH TAPER OFFSET ONLY ON MAINLINE (Non-Channelized)
Figure 6-2-B

6-2.03 Encroachment

Although there should desirably be no encroachment by a turning vehicle into the opposing lanes of the roadway onto which a vehicle is turning, the criteria in Table 6-2-C may not be attainable at some intersections. In these cases, consideration should be given to the encroachment that can be tolerated and the frequency for which encroachment is acceptable. Several factors may influence this assessment, including traffic volumes, truck traffic, functional classification, 1-way or 2-way operation, available roadway width, and right of way and environmental constraints.

Table 6-2-D provides guidelines for determining the encroachment for right turns that may be allowed for various conditions.

Table 6-2-D
GUIDELINES FOR ENCROACHMENT FOR RIGHT TURNS ONTO 2-LANE ROADWAYS

Turn Made From	Turn Made Onto	Acceptable Encroachment for Design Vehicle for Roadway Onto Which Turn is Made
Arterial	Arterial Collector Local	No encroachment into opposing lanes of travel 1 ft encroachment into opposing lanes of travel 2 ft encroachment into opposing lanes of travel
Collector	Arterial Collector Local	No encroachment into opposing lanes of travel 2 ft encroachment into opposing lanes of travel 3 ft encroachment into opposing lanes of travel
Local	Arterial Collector Local	No encroachment into opposing lanes of travel 3 ft encroachment into opposing lanes of travel 4 ft encroachment into opposing lanes of travel

Notes:

1. *See Tables 6-2-A and 6-2-B for the design vehicle selection. The encroachment guidelines refer to the selected design vehicle.*
2. *Before the turn is made, the design vehicle is assumed to be in the outermost through travel lane or exclusive right-turn lane, whichever applies. The turning vehicle is assumed to not encroach onto adjacent lanes on the roadway from which the turn is made.*
3. *When determining the acceptable encroachment, turning volumes, through volumes, and the type of traffic control at the intersection should be considered.*
4. *All proposed designs should be checked to ensure that the design vehicle can make the turn.*

6-2.04 Curb and Gutter

Curb and gutter should typically be provided along the turning radii of at-grade intersections as shown in Figure 6-2-A and Figure 6-2-B. The *Standard Drawings* include two types of curb and gutter with sloping curb faces that are more commonly used for placing along the radii — Type 2 and Type 3. However, the *Standard* Drawings also include a vertical curb (Type 1) that is sometimes used on urban roadways. The following information describes the application of these types of curb and gutter for rural and urban intersections.

Type 2 curb and gutter, which is four inches in height and is on a plane slope, is typically used along the radii of rural intersections. Type 2 curb and gutter should also be placed along the radii of urban intersections located within open shoulder sections of the mainline.

Type 3 curb and gutter, which has a round-faced curb with a height of six inches, is typically used in urban areas on low-speed roadways ($V \leq 45$ miles per hour). Type 1 curb and gutter, which has a vertical face with a height of six inches also, is preferred where sidewalks are proposed, especially if the full shoulder width and/or the minimum buffer width is not provided. In such cases where the mainline includes Type 1 or Type 3 curb and gutter, the continuation of the same type of curb and gutter is typically provided along the radii of the intersection, except where curb ramps are proposed.

Section 6-7.02.01 provides information about the types of curb and gutter to be used at channelized intersections. Section 14-2.06.01 provides more information on the different types and usage of curb and gutter along the mainline.

6-2.05 Other Intersection Factors

- On-Street Parking – On-street parking should be restricted before and after an intersection to accommodate large turning vehicles at intersections with small radii. As discussed in Section 6-2.03, some encroachment into the adjacent lane is allowable. The presence of an 8-foot or wider shoulder should have the same impact as a parking lane. Each intersection should be considered on a case-by-case basis. For more information, see Section 14-2.06.06 and Part 3 of the *MUTCD*.

- Pedestrians – Pedestrians may be affected by the selection of the turning radii. The greater the radius, the farther pedestrians are expected to walk in the roadway, which may lead to a decision to use a turning roadway to provide a pedestrian refuge. Typically, a pedestrian refuge island should be provided when the crossing distance exceeds 60 feet. Smaller turning radii (typically less than 30 feet) can substantially reduce the vehicular turning speed and reduce the open pavement area for pedestrians crossing the roadway; however, these radii may cause undesirable encroachments. Such trade-offs should be considered when designing an intersection. See Section 6-7.03 for more information on turning roadways. See the *Standard Drawings* for information on alignment of pedestrian curb ramps at intersections.

6-3.0 AUXILIARY LANES

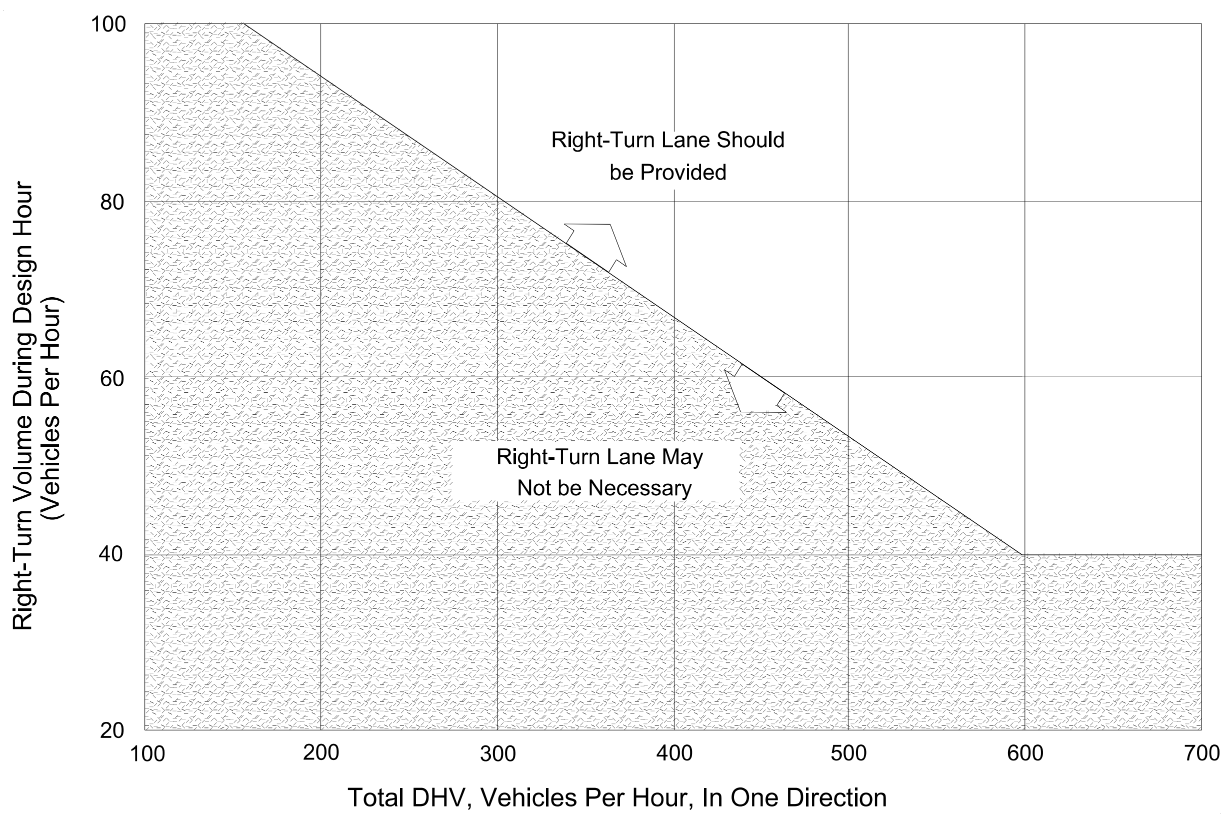
Drivers leaving a roadway at an intersection usually reduce their speed before turning. Drivers that enter a roadway at an intersection accelerate until the desired open-road speed is reached. Where this deceleration or acceleration occurs in the through travel lanes, the flow of through traffic is disrupted. To minimize this disruption, the use of auxiliary lanes may be warranted to improve the LOS of the roadway. The following subsections present information for determining when auxiliary lanes should be provided, and how lengths of auxiliary lanes should be determined.

6-3.01 Right-Turn Lanes

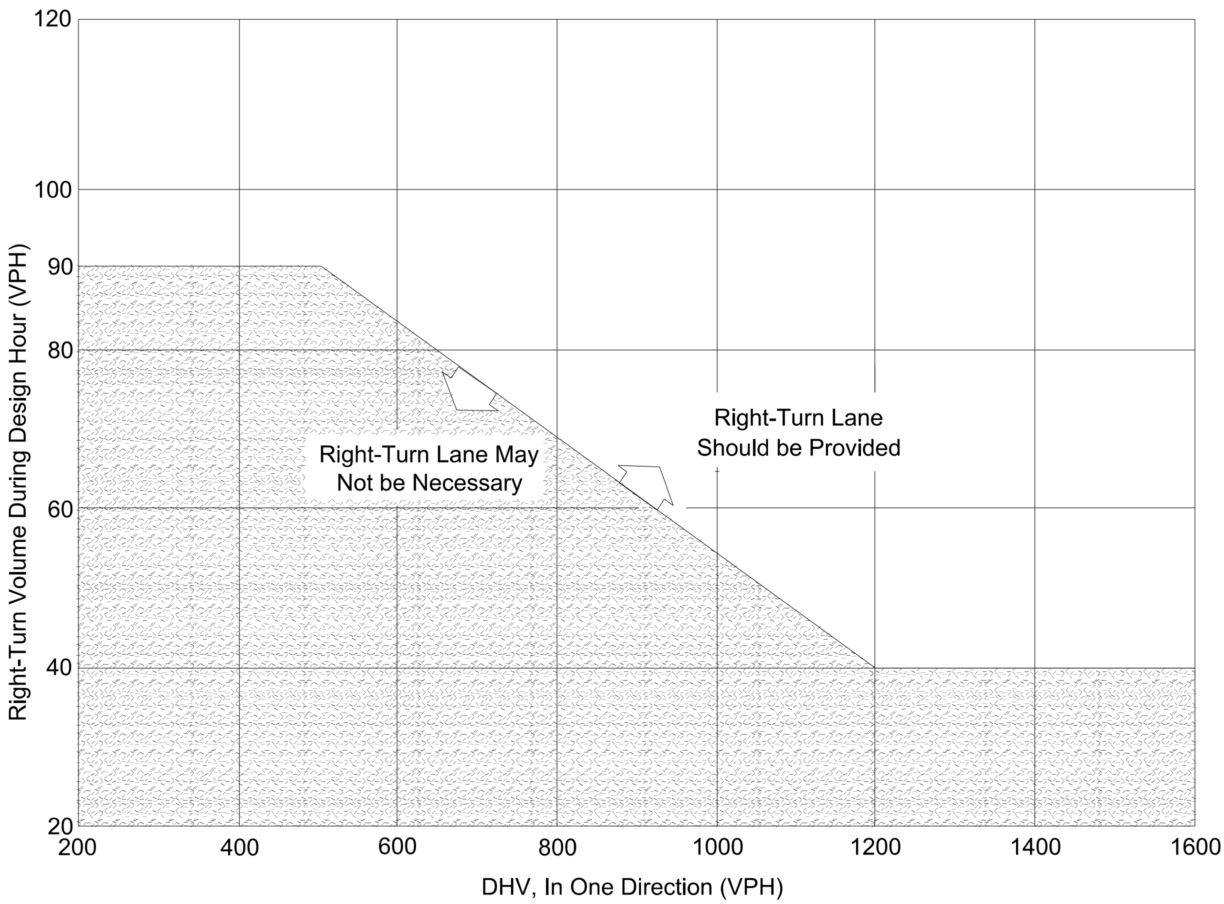
Exclusive right-turn lanes should be provided:

1. at any non-signalized intersection on a 2-lane urban or rural roadway that satisfies the criteria in Figure 6-3-A
2. at the free-flowing leg of any non-signalized intersection on a high-speed 4-lane urban or rural roadway (V ≥ 50 miles per hour) that satisfies the criteria in Figure 6-3-B
3. at any intersection where a capacity analysis determines a right-turn lane is recommended to meet the criteria for LOS
4. at any intersection with a crash pattern, with disruptions to traffic operations, with adverse geometrics (e.g., restricted sight distance), or if engineering judgment indicates the need for a separate, exclusive right lane
5. at any signalized intersection where the right-turn volume exceeds 300 vph and the adjacent through volume exceeds 300 vph per lane

For additional information, see NCHRP Report 457 *Evaluating Intersection Improvements*.



RIGHT-TURN LANES AT NON-SIGNALIZED INTERSECTIONS ON 2-LANE ROADWAYS
Figure 6-3-A



Note: Figure is only applicable on roadways with a design speed of 50 miles per hour or greater.

RIGHT-TURN LANES AT NON-SIGNALIZED INTERSECTIONS
ON 4-LANE ROADWAYS
Figure 6-3-B

6-3.02 Left-Turn Lanes

Exclusive left-turn lanes should be provided:

1. at median crossovers on divided urban and rural roadways with a median wide enough to accommodate a left-turn lane (see Section 6-8.02)
2. at 2-lane roadways with a design speed of 65 mph
3. at any non-signalized intersection on a 2-lane roadway that satisfies the criteria in Figures 6-3-C, 6-3-D, or 6-3-E
4. at any intersection where a capacity analysis determines a left-turn lane is recommended to meet the criteria for LOS
5. at any intersection with a crash pattern, with disruptions to traffic operations, with adverse geometrics (e.g., restricted sight distance), or if engineering judgment indicates the need for a separate, exclusive left turn lane

The following example illustrates how to use Figures 6-3-C through 6-3-E to determine if a left-turn lane should be provided based on volume:

Example 6-3-1

Given: Design Speed = 60 miles per hour
V_A = 400 vehicles per hour
V_O = 200 vehicles per hour
Left turns in V_A = 10%

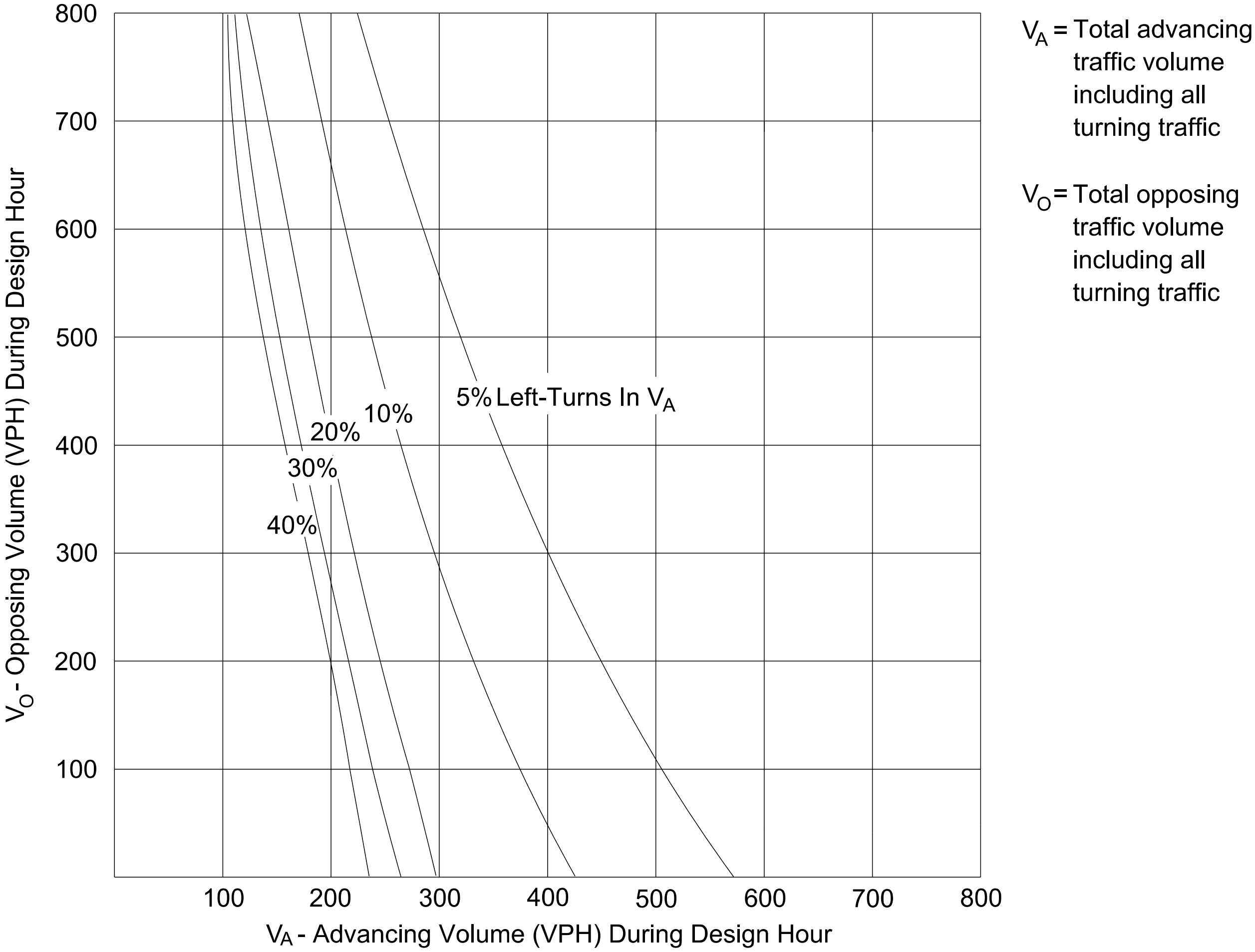
Problem: Determine if a left-turn lane is warranted.

Solution:

Step 1: Based on the left-turn volumes, the 10% curve in Figure 6-3-C should be used.

Step 2: The intersection point of V_A (400 vehicles per hour) and V_O (200 vehicles per hour) should be located in the chart.

Step 3: The intersection point determined in Step 2 is located to the right of the 10% curve in Figure 6-3-C; therefore, a left-turn lane should be provided.



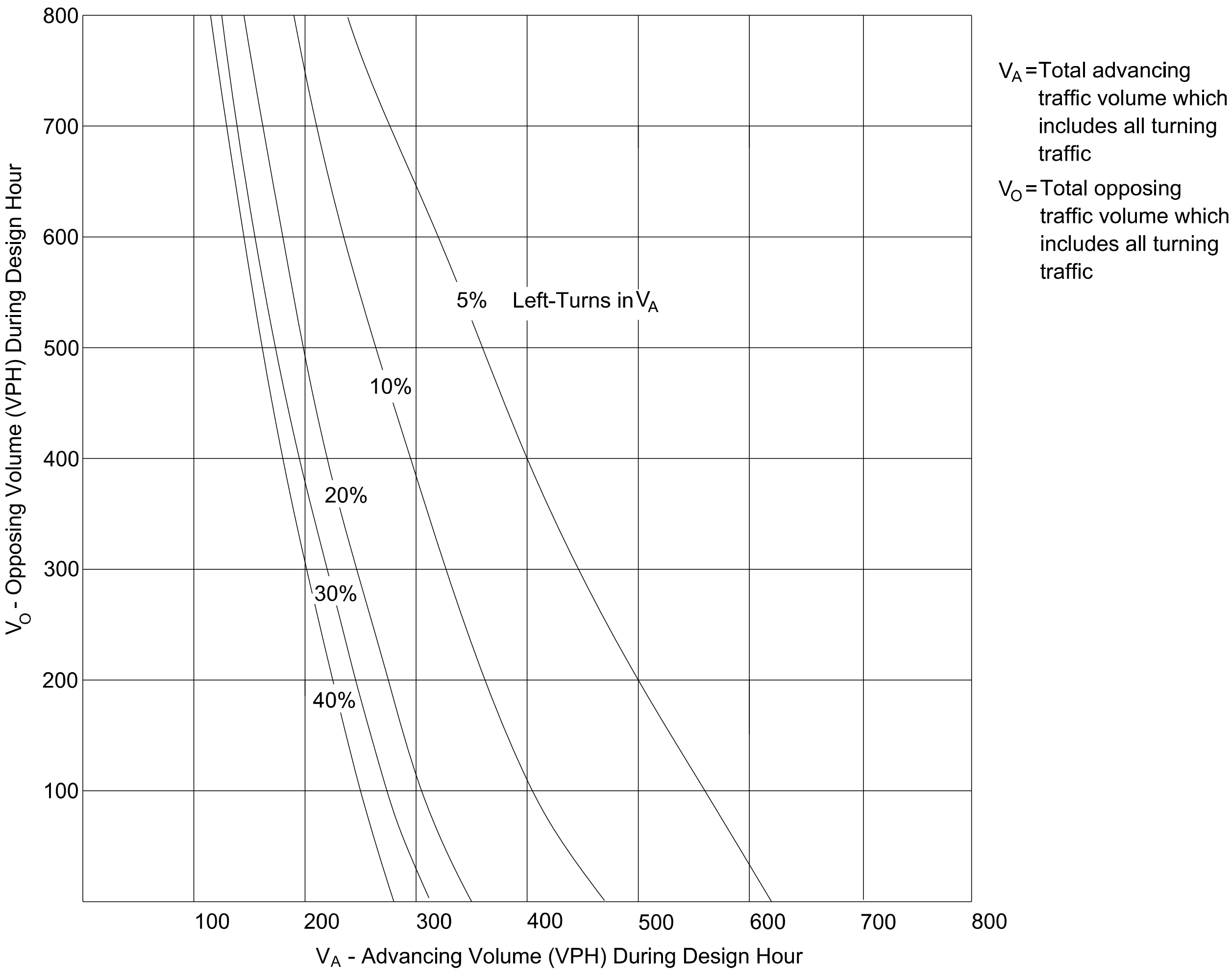
Instructions:

1. *The family of curves represents the percent of left turns in the advancing volume (V_A). The curve for the actual percentage of left turns should be located. When this is not an even increment of five, an estimate of where the curve lies should be made.*

2. *The intersection point of V_A and V_O should be located in the chart.*

3. *The location of the intersection point in Item 2 above relative to the curve in Item 1 above should be determined. If the point is to the right of the curve, then a left-turn lane should be provided. If the point is to the left of the curve, then a left-turn lane is generally not warranted based on traffic volumes.*

LEFT-TURN LANES AT NON-SIGNALIZED
INTERSECTIONS ON 2-LANE ROADWAYS (55 mph – 60 mph)
Figure 6-3-C



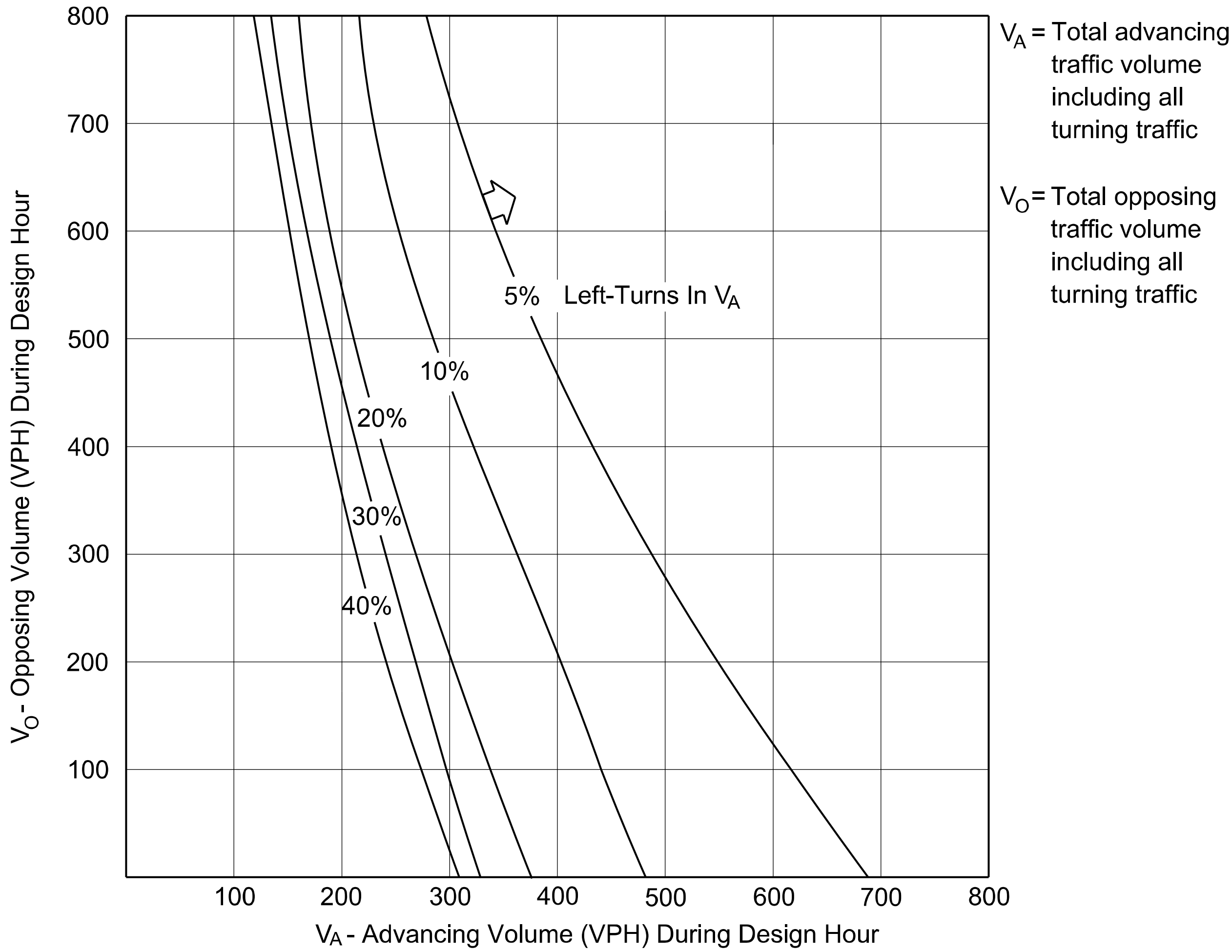
Instructions:

1. *The family of curves represents the percent of left turns in the advancing volume (V_A). The curve for the actual percentage of left turns should be located. When this is not an even increment of five, an estimate of where the curve lies should be made.*

2. *The intersection point of V_A and V_O should be located in the chart.*

3. *The location of the intersection point in Item 2 above relative to the curve in Item 1 above should be determined. If the point is to the right of the curve, then a left-turn lane should be provided. If the point is to the left of the curve, then a left-turn lane is generally not warranted based on traffic volumes.*

LEFT-TURN LANES AT NON-SIGNALIZED INTERSECTIONS ON 2-LANE ROADWAYS (45 mph – 50 mph)
Figure 6-3-D



Instructions:

1. *The family of curves represents the percent of left turns in the advancing volume (V_A). The curve for the actual percentage of left turns should be located. When this is not an even increment of five, an estimate of where the curve lies should be made.*

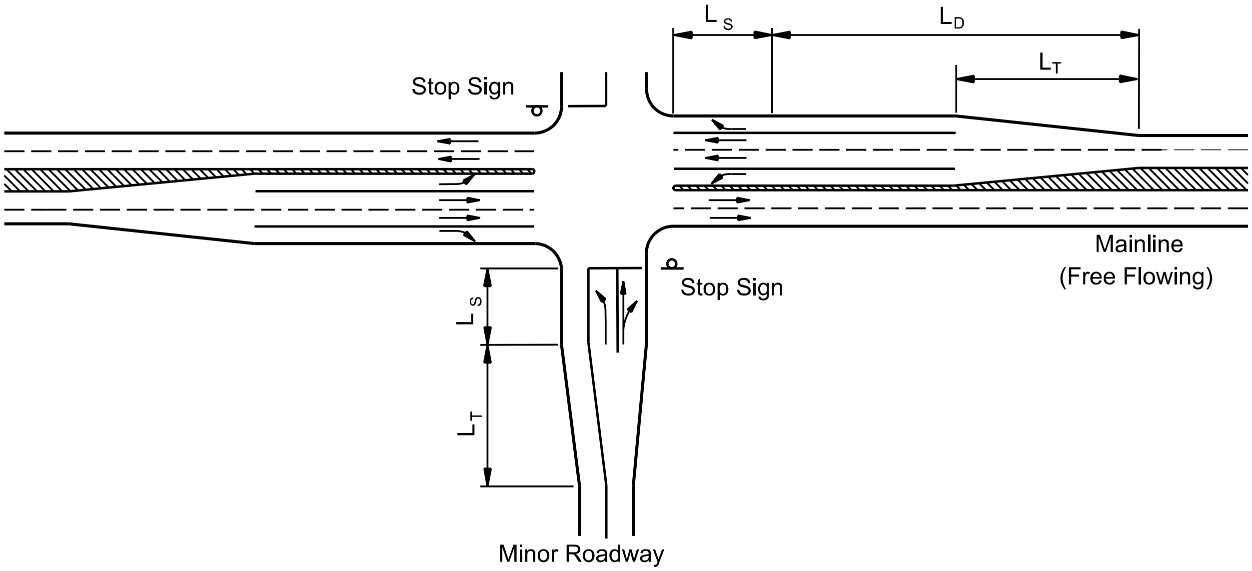
2. *The intersection point of V_A and V_O should be located in the chart.*

3. *The location of the intersection point in Item 2 above relative to the curve in Item 1 above should be determined. If the point is to the right of the curve, then a left-turn lane should be provided. If the point is to the left of the curve, then a left-turn lane is generally not warranted based on traffic volumes.*

LEFT-TURN LANES AT NON-SIGNALIZED INTERSECTIONS ON 2-LANE ROADWAYS (≤ 40 mph)
Figure 6-3-E

6-3.03 Length of Auxiliary Turning Lanes

The length of a right-turn or left-turn lane at an intersection should allow safe deceleration and storage of turning vehicles outside of the through lanes, thereby improving safety and the LOS. Figure 6-3-F illustrates typical auxiliary lanes and their segments at an intersection. As shown in the figure, the length of auxiliary lanes consists of a combination of the deceleration length (L_D) (which includes the taper length (L_T)) and the storage length (L_S). The following subsections discuss the application of these elements to determine the minimum length of an auxiliary turn lane based on design speeds and traffic volumes.



Key: L_T = *Taper Length*
L_D = *Deceleration Length*
L_S = *Storage Length*

Note: The schematic of the mainline (free flowing) also applies to all legs of a signalized intersection. In such cases, the auxiliary lane along the crossing route should be determined based on the design speed and traffic volume of the crossing route.

TYPICAL AUXILIARY LANES AT AN INTERSECTION
Figure 6-3-F

6-3.03.01 Taper Length

The taper length (L_T) for deceleration lanes should typically be 150 feet. A shorter taper length may be used at restricted sites; however, the minimum taper length should be 75 feet.

6-3.03.02 Application of Deceleration Length

Table 6-3-A presents criteria for the application of deceleration lengths for turn lanes at an intersection. The appropriate storage length (L_S) should be added to the deceleration length (L_D) to obtain the full length for design of the turning lane. Reduced lengths may be more feasible on roadways with low traffic volumes or those that are located in restricted areas; however, the overall length of the auxiliary lane should not be less than 200 feet.

At intermediate median crossovers (crossovers without an intersecting crossing route), the full deceleration length is not warranted; instead, a 150-foot taper and 150-foot turn lane should typically be used as shown in Figure 6-8-D.

Table 6-3-A
LENGTHS OF TURNING LANES FOR DECELERATION (L_D)

Mainline Design Speed (mph)	L_D (ft)
30	90*
35	120*
40	155*
45	200
50	240
55	290
60	345
65	405

* *The taper length should typically be 150 feet. A shorter taper length may be used in restricted sites; however, the minimum taper length should be 75 feet. Therefore, these lengths should be used only in restricted sites.*

Notes:

1. *The values in the table assume that deceleration starts as the vehicle enters the taper. The initial speed is equal to the design speed.*
2. *L_D values from the table include the taper length.*
3. *L_D values from the table do not include the storage length.*
4. *The L_D component of the turn lane only applies to approaches to non-signalized intersections. Stop-controlled approaches to an intersection only include the storage length and the taper length.*

6-3.03.03 Storage Length

Where a turn lane is used, the storage length (L_S) should be sufficient to store the number of vehicles likely to accumulate in the design hour. Recommended storage length criteria are as follows:

1. Signalized Intersections – Roadway Design Division should consult with the Traffic Engineering Division and the Planning Division to determine the recommended storage lengths at signalized intersections.
2. Non-signalized Intersections – The storage length should be based on the number of turning vehicles likely to arrive within the design hour. Table 6-3-B provides recommended

storage lengths for right- and left-turn lanes at a non-signalized intersection based on the hourly turning volume.

Table 6-3-B
RECOMMENDED STORAGE LENGTHS (L_S) FOR NON-SIGNALIZED TURN LANES

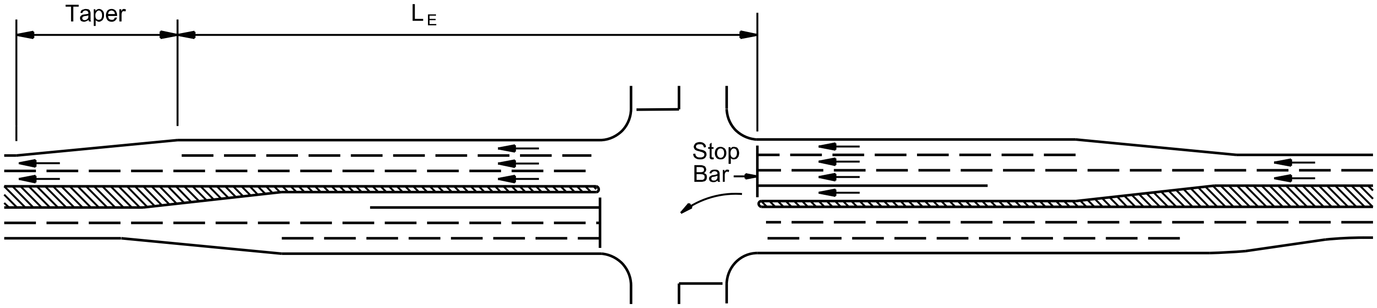
DHV (vph)	L_S (ft)
≤ 60	50 – 75
61 – 120	100
120 – 180	150
> 180	200 or greater

3. Minimum Length – The storage length for turn lanes should be sufficient to store a minimum of two vehicles, which should typically be 50 feet. However, where the turning truck traffic is greater than 10%, the intersection should be analyzed by Planning Division to determine the appropriate length.

6-3.03.04 Design Length for Extension of Through Lanes

Projects that widen roadways to provide additional capacity may terminate at an intersection, especially in urban areas. In such cases, the additional through lanes are either dropped at the intersection or carried through the intersection and transitioned to the width of the existing roadway. Additionally, through lanes are sometimes added at an intersection to meet the criteria for LOS, especially at signalized intersections, as shown in Figure 6-3-G.

Any additional through lanes, or combination turn and through lanes, should be extended beyond the intersection a sufficient distance for vehicles to return to the through lanes without affecting the roadway operation. Figure 6-3-G provides the recommended lengths (L_E) for vehicles to return to the through lane beyond the stop bar. The recommended taper length is 150 feet for low-speed roadways ($V \leq 45$ miles per hour) and 300 feet for high-speed roadways ($V \geq 50$ miles per hour).



Design Speed (mph)	L_E (ft)
30	160
35	215
40	320
45	430
50	585
55	780
60	1010
65	1300

Notes:

1. *L_E is the distance for a vehicle to accelerate from a stop to five miles per hour below the design speed.*

2. *The taper distance should be 150 feet for low-speed roadways and 300 feet for high-speed roadways.*

EXTENSION OF ADDITIONAL THROUGH LANES
Figure 6-3-G

6-3.03.05 Additional Design Considerations

The following factors should be considered when designing an auxiliary turn lane:

1. Channelized Left-Turn Lane – If a left turn is proposed on a 2-lane roadway, consideration should be given to designing it as a fully channelized left-turn lane. A typical channelized left turn lane is illustrated in Figure 6-3-H. Left-turn deceleration and storage bays for 2-lane roadways should typically be designed symmetrically about the roadway centerline. However, right of way or other constraints may result in an asymmetrical design.

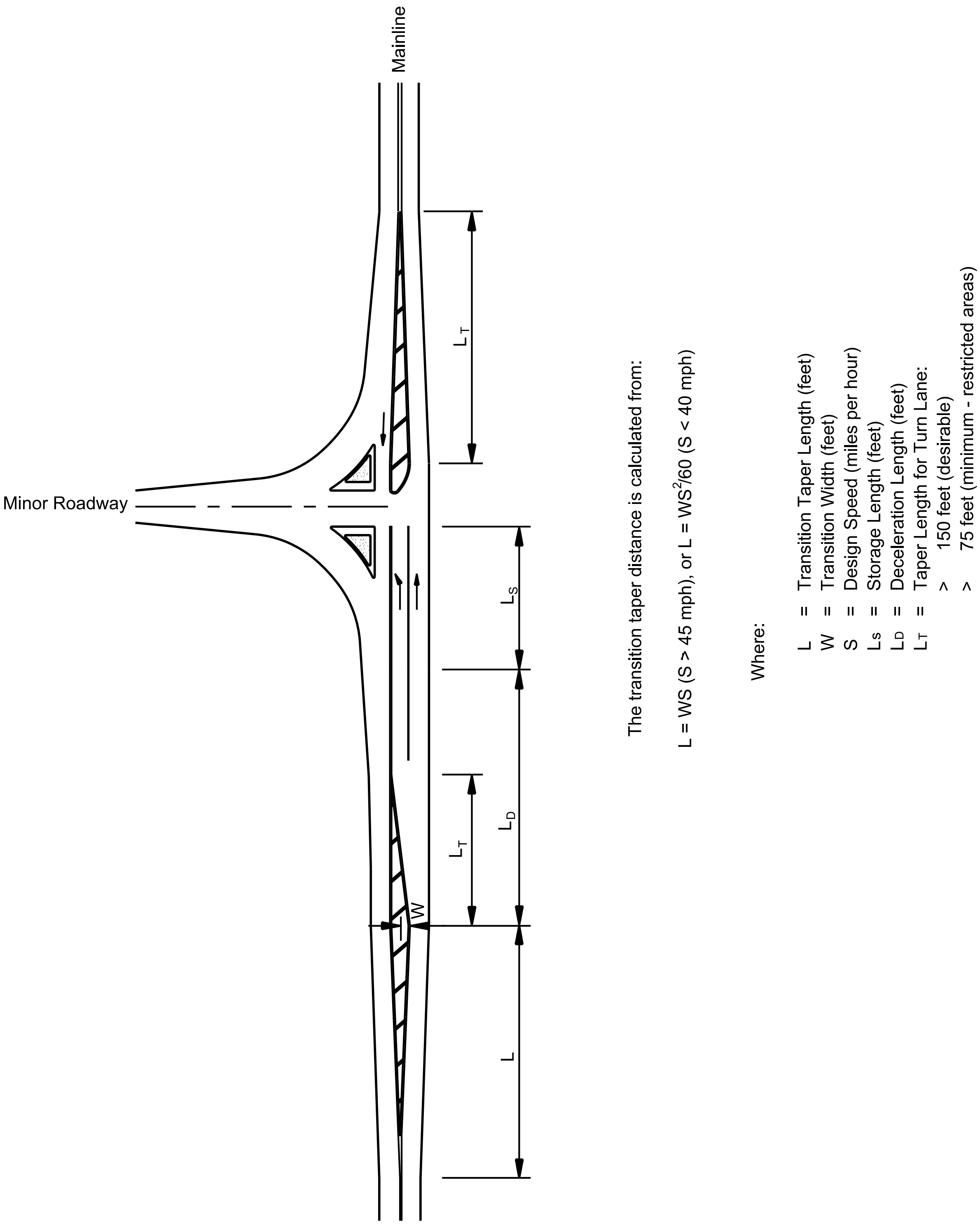
2. Offset Left-Turn Lanes – Where median width in a narrow-median section is adequate, the left-turn lanes should preferably be aligned so they are offset to the left of each other. The advantages of offsetting the left-turn lanes are:

   a. the sight line for opposing through traffic for each left-turning vehicle is not blocked by the opposing left-turning vehicle

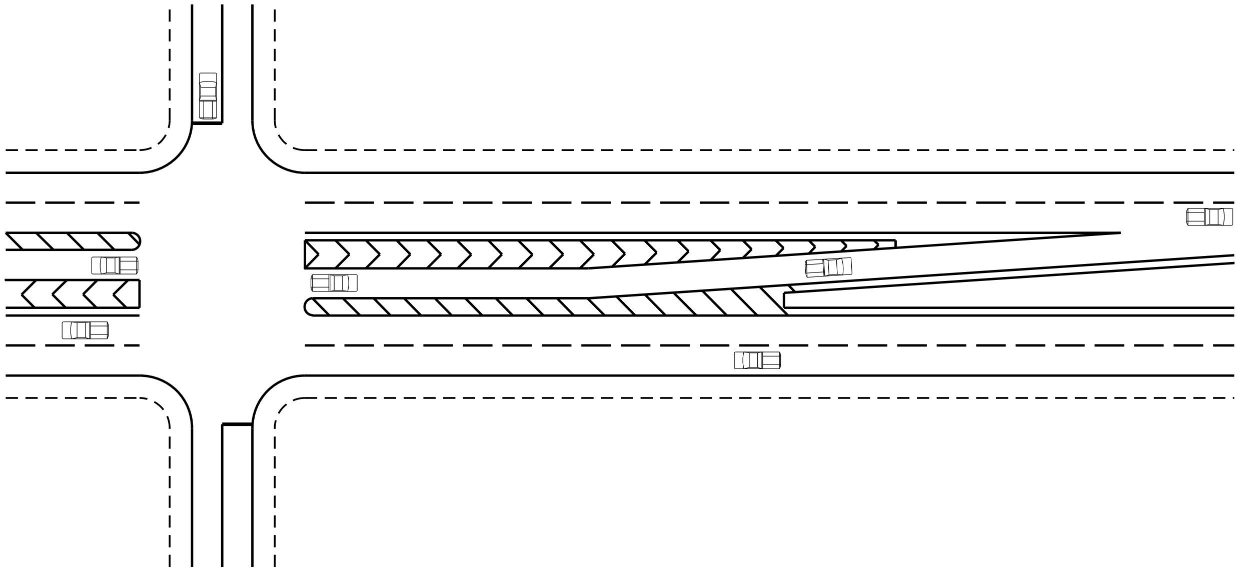
   b. decreased probability of a conflict between opposing left-turn movements within the intersection

   c. more left-turn vehicles served in a given period of time, especially at a signalized intersection

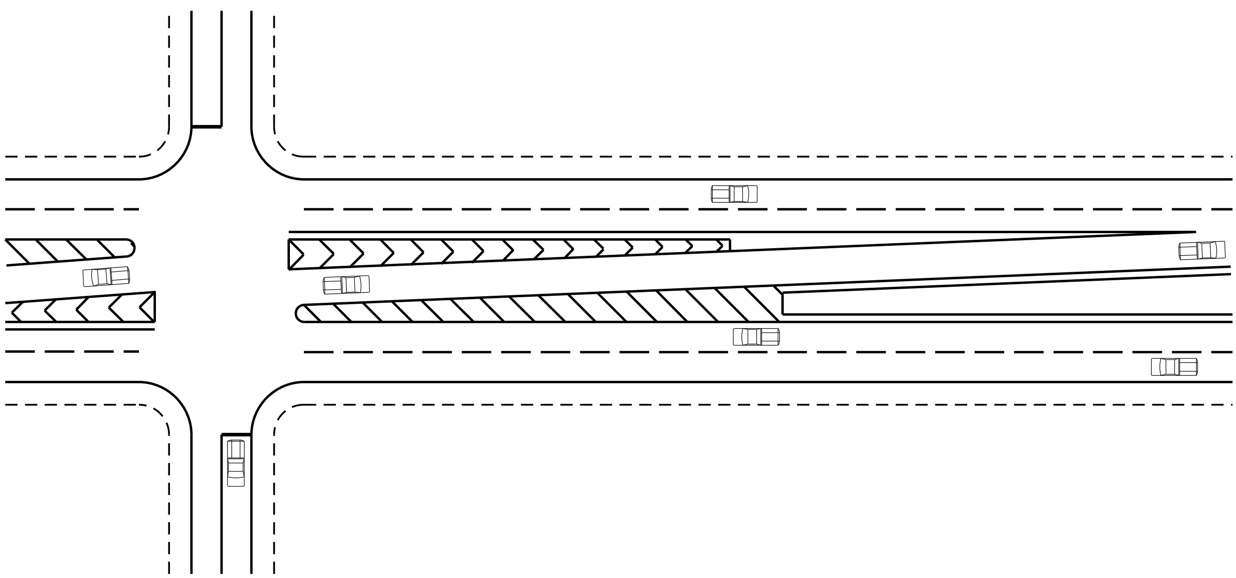
   Offset designs may be a parallel or a taper design as shown in Figure 6-3-I. Offset turn lanes should be separated from the adjacent through traveled way by a divider that should desirably be no less than four feet wide.



CHANNELIZED LEFT-TURN LANE FOR 2-LANE ROADWAYS
Figure 6-3-H



(a) Parallel-Offset Turn Lane



(b) Tapered-Offset Turn Lane

OFFSET LEFT-TURN LANES
Figure 6-3-I

3. Parking Restrictions – A right-turn lane in an urban area may warrant parking restrictions beyond the usual restricted distances from the intersection, as discussed in Sections 6-2.05 and 14-2.06.06.

4. Width – The width of the turn lane should be determined based on the functional classification and urban/rural location of the intersection. See the geometric design criteria tables in Chapters 2, 12, and 14 for more information.

5. Intersection Sight Distance (ISD) – Section 6-6.0 provides information on ISD criteria.

6. Median Crossovers – Section 6-8.0 provides information on the design of median crossovers.

7. Pavement Markings – Typical pavement markings of a left-turn lane are illustrated in the *Standard Drawings*. The *MUTCD* should also be referenced to determine the proper pavement markings for unique cases.

6-3.04 Dual Turn Lanes

6-3.04.01 Warrants

Dual right- and/or left-turn lanes should be considered at signalized intersections if any of the following situations exist:

1. space is insufficient to provide the recommended length of a single turn lane because of restrictive site conditions (e.g., closely spaced intersections)

2. the recommended length of a single-turn lane becomes prohibitive

3. 300 or more left-turning vehicles in the design hour

4. the time for a protected left-turn phase for a single lane becomes unattainable when trying to meet the criteria for LOS

Consideration of dual right turns should be analyzed for each specific situation with regard to traffic demand and the geometrics. If feasible, an alternative means (e.g., channelization with a turning roadway) should be considered to accommodate a high number of right-turning vehicles (see Section 6-7.0).

The lower efficiency of dual right-turn lanes, as compared to dual left-turn lanes, is reflected in the capacity methodology for signalized intersections in the *Highway Capacity Manual.*

6-3.04.02 Design

For dual turn lanes to work properly, several design elements should be evaluated. Figure 6-3-J illustrates the key design elements for dual right- and left-turn lanes. The following information should be applied:

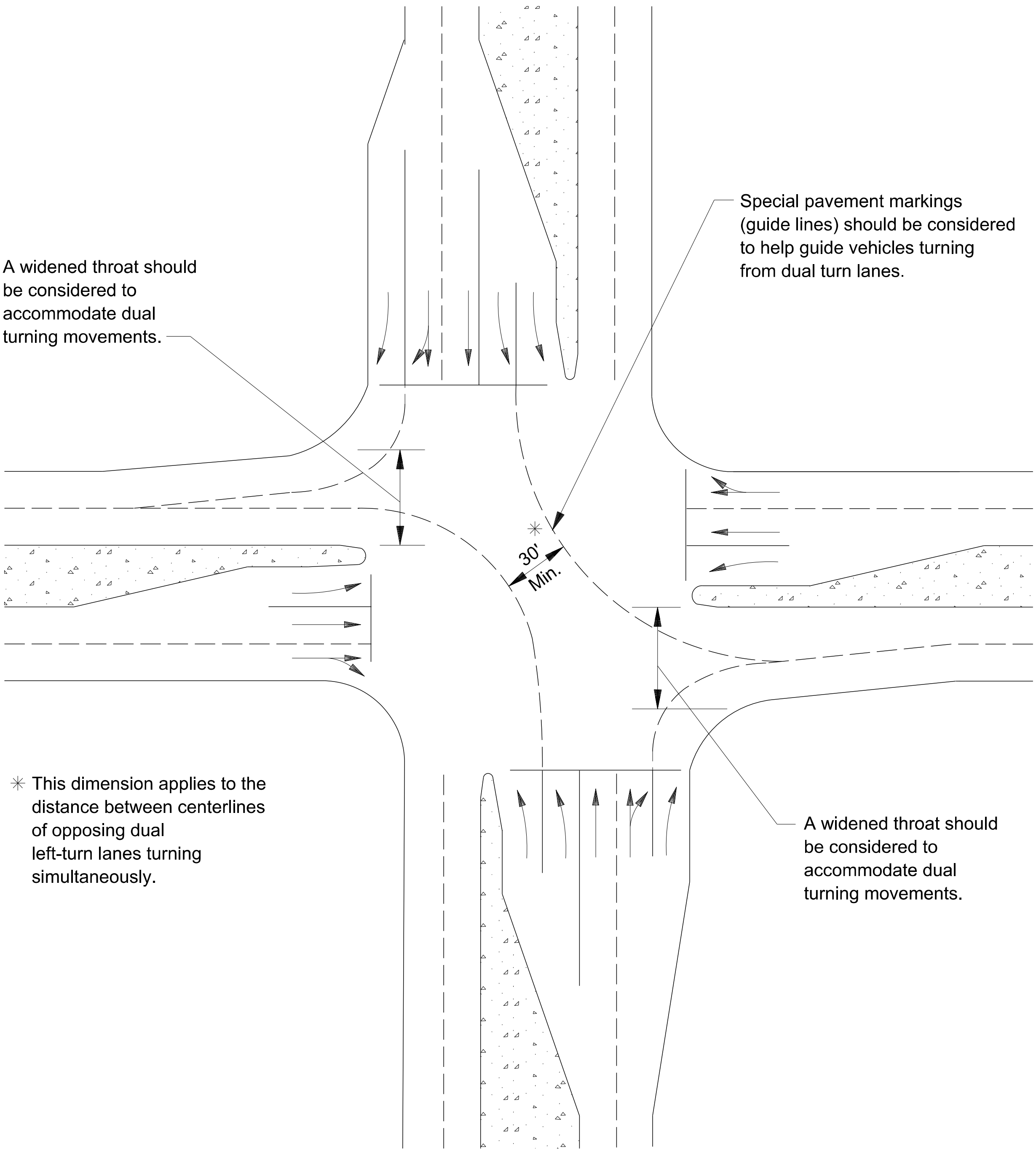
1. Entry Tapers – The taper rate should be no steeper than 8:1 for design speeds up to 30 miles per hour, 12:1 for design speeds of 35-45 miles per hour, and 15:1 for design speeds of 50 miles per hour and greater.

2. Opposing Left-Turn Traffic – Where simultaneous opposing dual left turns are allowed, the distance between opposing left-turning vehicles should be checked to ensure that sufficient space is provided for all turning movements. The minimum distance between the centerlines of opposing flows of left-turning traffic should be 30 feet. See Figure 6-3-J for the measurement of the 30-foot dimension. Intersection layouts should be coordinated with the Traffic Engineering Division.

3. Design Vehicles – All intersection design elements for dual turn lanes should be checked to ensure that the design vehicle can make the turn. The selected design vehicle should be assumed to be turning from the outside lane of the dual turn lane. The inside vehicle should desirably be an SU-40, but as a minimum, should be a passenger vehicle turning side by side with the selected design vehicle.

4. Throat Width – Because of the off-tracking characteristics of turning vehicles, the normal width of two travel lanes may be inadequate to properly receive two vehicles turning simultaneously. The throat width should be checked to ensure that the design vehicle has adequate room to turn. The receiving width for dual left-turn lanes should typically be 30 feet to 36 feet. For dual right-turn lanes, a 36-foot throat width is generally recommended. When determining the available throat width, the width of the full-depth paved shoulder, if present, may be used to accommodate dual turns.

5. Pavement Marking – As illustrated in Figure 6-3-J, pavement markings should be considered to effectively guide two lines of vehicles turning side by side. The Traffic Engineering Division should be consulted to determine the proper location of pavement markings for dual turn lanes.



SCHEMATIC OF DUAL TURN LANES
Figure 6-3-J

6-4.0 CONTINUOUS TWO-WAY LEFT-TURN LANES

Continuous Two-Way Left-Turn Lanes (CTWLTL) are one alternative to accommodate a continuous left-turn demand. The information presented in the following sections should be evaluated when considering the implementation of a CTWLTL.

6-4.01 Guidelines for Application

Each location should be analyzed to verify the need and feasibility of a CTWLTL. The applicability of the CTWLTL should be a function of the traffic conditions resulting from the adjacent land use. The surrounding area should be evaluated to determine the relative benefits of a CTWLTL as compared to alternative access techniques, such as a raised median to create a divided section. A CTWLTL may lead to additional strip development; therefore, a raised median may be preferred.

For traffic volumes greater than 20,000 ADT in the design year, studies should be conducted to determine if a raised median design is more appropriate than a CTWLTL. The extent of development, number of traffic signals, number of driveway entrances, gaps available in the oncoming traffic, the number of through lanes, and the use of dual left-turn lanes at intersections should be considered to determine the appropriate selection.

6-4.02 Design Speed

The design speed on a roadway is a key factor for consideration in CTWLTL applications. General criteria dictates that low-speed roadways ($V \leq 45$ miles per hour) should accommodate a CTWLTL. High-speed roadways ($V \geq 50$ miles per hour) where a CTWLTL is being considered should be investigated for factors that may include, but are not limited to, safety, the amount of left-turning traffic, and traffic signals.

6-4.03 Lane Width

Lane widths for a proposed CTWLTL are presented in Table 6-4-A.

Table 6-4-A
LANE WIDTHS FOR A CTWLTL

Desirable	Minimum
14 feet	12 feet

Notes:

1. *In industrial areas with heavy truck traffic, the CTWLTL width should desirably be 16 feet.*
2. *For new construction projects on new location, the minimum width should be 14 feet.*

Existing roadways where a CTWLTL is being considered are often located in areas of restricted right of way, thereby making it difficult to add the CTWLTL. Such projects are subject to new construction/reconstruction criteria (if the project is adding capacity to the roadway) or 3R criteria (if the project is not adding capacity to the roadway). For projects that are subject to new construction/reconstruction criteria, 3R criteria may be used for the cross-section elements (e.g., lane width, shoulder width) in order to minimize the right of way impacts.

For projects that are subject to 3R criteria, the following alternatives may be considered to minimize right of way impacts:

1. reduce the width of existing through lanes (to the minimum 3R width)
2. reduce the number of existing through lanes
3. eliminate existing parking lanes
4. reduce the width of existing shoulders (to the minimum 3R width)

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

6-4.04 Intersection Treatment

At intersections with public roads, a CTWLTL should be 1) terminated in advance of the intersection to allow the development of an exclusive left-turn lane, or 2) extended to the intersection area. The preferred treatment is to terminate the CTWLTL and provide an exclusive left-turn lane; otherwise, drivers may pass through the intersection in the CTWLTL to make a left turn just beyond the intersection. The following factors should be considered when determining whether to terminate or extend the CTWLTL at intersections:

1. Signalized Intersections – A CTWLTL should be terminated in advance of the intersection because signalized intersections typically warrant an exclusive left-turn lane.

2. Non-signalized Intersections – Where left-turn volumes are low, the CTWLTL may be extended through an intersection where an exclusive left-turn lane is not warranted. Heavy left-turn volumes may indicate that an exclusive left-turn lane is warranted.

3. Minimum Length of CTWLTL – The intersection treatment may affect the minimum length of the CTWLTL between intersections. The CTWLTL length may be influenced by through traffic volumes, turning volumes, and operating speeds on the roadway. The following guidelines should be used:

    a. On roadways with lower speeds and/or lower traffic volumes, the minimum length of a CTWLTL between intersections should preferably be 500 feet.

    b. On roadways with higher speeds and/or higher traffic volumes, the minimum length of a CTWLTL between intersections should preferably be 1000 feet.

4. No Access ROW – A CTWLTL should not be used within No Access ROW limits, such as within the limits of an interchange. Section 6-7.04 and Chapter 7, “Grade Separations and Interchanges”, provide more information on design of intersections within No Access ROW limits.

6-4.05 Traffic Control

A CTWLTL should include proper signing and marking to reduce indecision and misuse. Criteria for signing and marking are presented in the *MUTCD* and in the *Standard Drawings*.

6-5.0 ROUNDABOUTS

6-5.01 Roundabouts

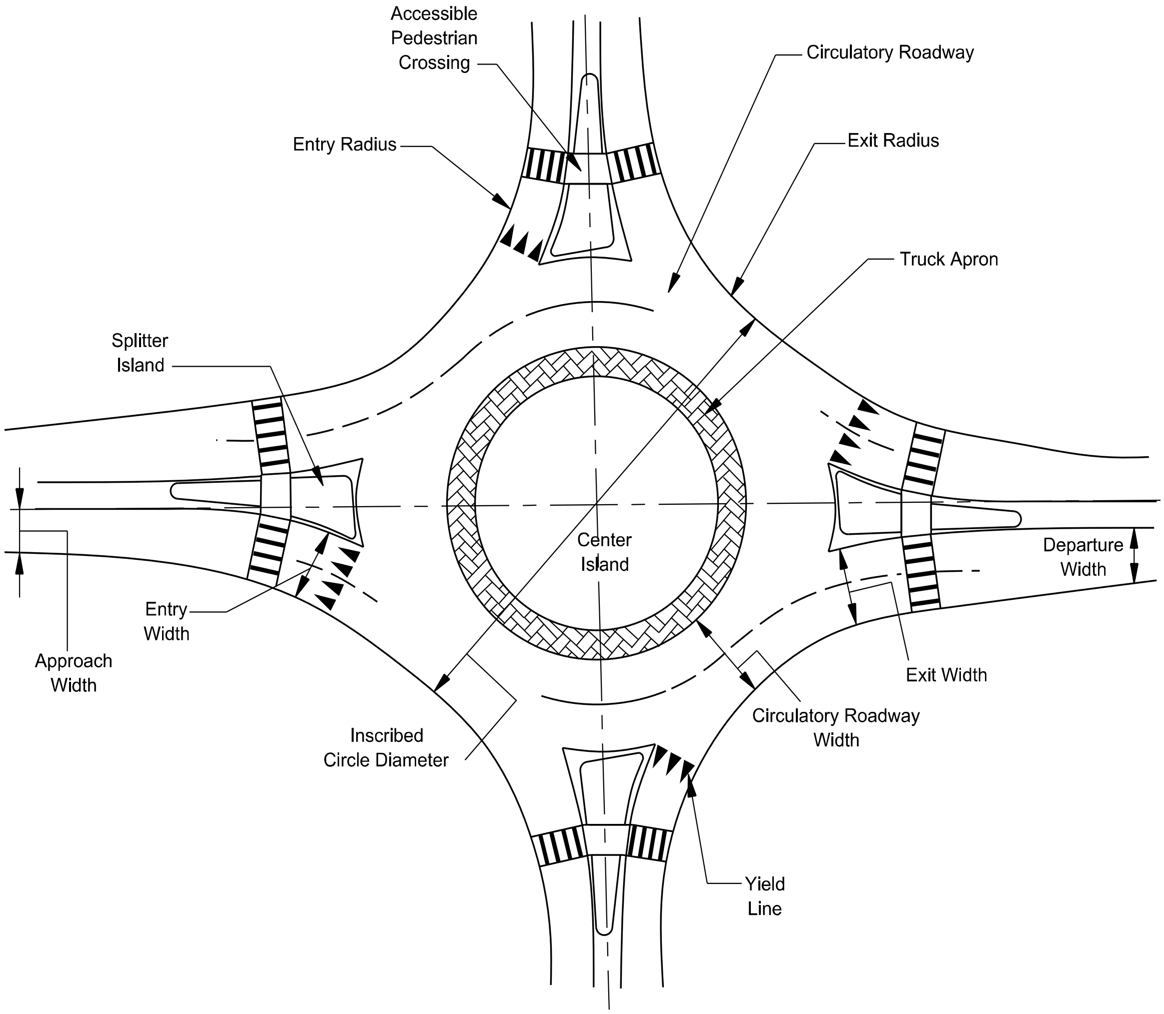
Roundabouts are circular intersections with specific design and traffic control features. These features include yield control of all entering traffic, channelized approaches, and geometric curvature and features to induce desirable vehicular speeds (typically less than 30 miles per hour). Figure 6-5-A illustrates the key components of a roundabout.

6-5.02 Intersection Type Selection

In determining whether to use a roundabout or a more traditional intersection at a site, the following should be considered:

- Safety – The frequency of crashes at an intersection is related to the number of conflict points at an intersection, as well as the magnitude of conflicting flows at each conflict point. A conflict point is a location where the paths of two vehicles diverge, merge, or cross each other. For example, the number of vehicle-vehicle conflict points for 3-legged intersections drops from 32 to eight with roundabouts — a 75% decrease. Fewer conflict points mean fewer opportunities for collisions. In addition, a roundabout has zero vehicle crossing points.

  The severity of a collision is determined largely by the speed and angle of impact. The higher the speed or angle of impact, the more severe the collision. Roundabouts reduce or eliminate many conflicts that are present in traditional intersections.



ROUNDABOUT ELEMENTS
Figure 6-5-A

- Movements – The overall roadway classification system and hierarchy should be considered before selecting a roundabout or stop-controlled intersection. Roundabouts tend to treat all movements at an intersection equally. Each approach should yield to circulating traffic, regardless of whether the approach is a local roadway or major arterial. In other words, all movements are given equal priority, which may result in more delay to the major movements than might otherwise be desired. Delays depend on the volume of turning movements and should be analyzed individually for each approach.

- Right of Way – Roundabouts usually require more space for the circular roadway and central island than the rectangular space inside traditional intersections. Therefore, a roundabout may have a significant right of way impact on the corner properties at an intersection, especially when compared with other types of intersections. The dimensions of a traditional intersection are typically comparable to the envelope formed by the approaching roadways. However, to the extent that a comparable roundabout would outperform a signal in terms of reduced delay and thus shorter queues, a roundabout should result in less queue storage space on the approach legs. As a result, roundabouts may reduce the need for additional right of way on the links between intersections at the expense of additional right of way at the intersections themselves. The right of way savings between intersections may make it feasible to accommodate parking, wider sidewalks, planter strips, wider outside lanes, and/or bicycle lanes in order to better accommodate pedestrians and/or bicyclists.

At existing interchange ramp terminals, paired roundabouts have been used to better accommodate left turns from the exit ramps and left turns onto the entrance ramps.

6-5.03 Locations

Roundabouts may be considered at locations where one or more of the following apply:

1. intersections with high-crash rates/high-severity rates
2. intersections with complex geometry (e.g., more than four approaches)
3. rural intersections with high-speed approaches (V ≥ 50 miles per hour)
4. interchange ramp terminals
5. closely spaced intersections
6. replacement of all-way stops
7. replacement of signalized intersections
8. intersections with high left-turn volumes
9. replacement of 2-way stops with high side-roadway delay
10. intersections with high U-turn movements
11. transitions from higher-speed to lower-speed areas

Roundabouts are not appropriate everywhere. Intersections that may not be good candidates for roundabouts include those with topographic or site constraints that limit the ability to provide appropriate geometry, those with highly unbalanced traffic flows (i.e., high traffic volumes on the main roadway and light traffic on the side roadway), and isolated intersections in a network of traffic signals.

6-5.04 Design Considerations

This subsection contains several of the key elements for designing roundabouts, and Table 6-5-A provides some of the controlling features. For more on the latest design guidance on roundabouts, the information provided in NCHRP Report 672 *Roundabouts: An Informational Guide – Second Edition* and guidance found on the FHWA's roundabout website should be reviewed.

1. Volumes – In general, a roundabout capacity analysis is not warranted if the total volume entering a 4-legged roundabout is less than 10,000 vehicles per day for 1-lane roundabouts and 20,000 vehicles per day for 2-lane roundabouts. For 3-legged roundabouts, 75% of above volumes should be used. Volumes above these amounts do not automatically warrant increasing the size of the roundabout.

Table 6-5-A
ROUNDABOUT CONTROLLING FEATURES

Design Element	Single-Lane Roundabout	Dual-Lane Roundabout
Maximum entry design speed	25 mph	30 mph
Maximum number of entering lanes per approach	1	2
Typical inscribed circle diameter	90 to 180 ft	150 to 220 ft
Central island treatment	Raised (with traversable apron)	Raised (with traversable apron)
Typical daily service volumes on 4-legged roundabout *	Up to approximately 25,000 (veh/day)	Up to approximately 45,000 (veh/day)

* *Operational analysis recommended to verify upper limit for specific applications or for roundabouts with more than two lanes or four legs.*

The Traffic Engineering Division and/or Planning Division may recommend a capacity analysis for any proposed roundabout, especially if any of the following conditions exist:

a. the number of entry lanes is not the same for all legs
b. the volumes on the legs are not balanced
c. a high percentage of left-turn movements (over 30%)
d. a high volume of pedestrians
e. other geometric considerations that warrant additional analysis (e.g., nearby driveways or intersections)

If a capacity analysis is recommended, the roundabout and each approach leg of the roundabout should operate at no more than 85% of capacity (0.85 maximum degree of saturation). For single-lane roundabouts, maximum volumes should be 1500 vehicles per hour (entry and circulating flow) and 1200 vehicles per hour (exit flow).

2. Speeds – The operating speed of a roundabout is widely recognized as one of its most important attributes in terms of safety performance; therefore, additional attention should be given to the design speed of a roundabout. Maximum entering design speeds should be 25 miles per hour for single-lane roundabouts or 30 miles per hour for multilane roundabouts. Speeds are influenced by a variety of factors, including the geometry of the roundabout and the operating speeds of the approaching roadways.

Roundabouts on high-speed roadways ($V \geq 50$ miles per hour) may not be expected by approaching drivers. Therefore, the geometric design should include features that encourage drivers to decelerate before reaching the roundabout. For example, a series of progressively sharper curves on a high-speed roundabout approach helps slow traffic to an appropriate entry speed.

3. Traffic Control – Vehicles entering the roundabout should yield to the traffic within the circle. A YIELD sign should be placed at the entry along with the appropriate pavement markings.

4. Splitter Islands – Splitter Islands are designed to separate entering and exiting traffic. A properly designed splitter island also aids in reducing speeds entering the circulatory roadway and provides a refuge for pedestrians.

5. Central Island – The central island of a roundabout is a raised, mainly non-traversable area surrounded by the circulatory roadway. A circular central island is preferred because the constant-radius circulatory roadway helps promote constant speeds around the central island. Statues and other structures should not be located within the central island, thus discouraging pedestrian use.

6. Landscaping – The central island should preferably be landscaped to enhance driver recognition of the roundabout upon approach, break the headlight glare of oncoming vehicles, and obscure the line of sight through the roundabout to encourage slow speeds. However, the landscaping elements, at full growth, should not block the necessary sight distance for drivers to make decisions while maneuvering through the roundabout. Therefore, trees and shrubs may be placed on the inner part of the central island, but the perimeter portion of the central island should consist of low-level landscaping, such as shrubs, grass, or groundcover, that does not exceed two feet in height.

7. Truck Apron – A roundabout may include a traversable truck apron along the perimeter of the central island to accommodate larger design vehicles, while the circulatory roadway remains relatively narrow to adequately constrain the speed of smaller vehicles. To discourage passenger vehicles from using a truck apron, the apron should be designed with the outer edge having a sloped curb that is raised approximately two to three inches above the circulatory roadway surface. The apron should also be constructed of a different material than the pavement to differentiate it from the circulatory roadway. Passenger buses should be accommodated within the circulatory roadway without tracking over the truck apron.

8. Signing and Markings – Proper regulatory control, advance warning, and directional guidance should be used to enhance and support driver expectations. Signs should be located where they have the maximum visibility for road users, but a minimal likelihood of obscuring pedestrians and bicyclists.

9. Pedestrians – Pedestrian accommodations should be provided at roundabouts where pedestrians utilize the public right of way. Designers should refer to the current version of the *PROWAG* for *ADA* considerations at roundabouts.

10. Bicyclists – Bicyclists should be considered in roundabout design, especially in areas with moderate to heavy bicycle traffic. However, bicycle lanes should be terminated a minimum 100 feet in advance of roundabouts so that the bicyclists merge into traffic for appropriate circulation with other vehicles. Roundabouts slow drivers to speeds more compatible with bicycle speeds, while reducing high-speed conflicts and simplifying turn movements for bicyclists. Typical on-road bicyclist speeds are 12-20 miles per hour, so designing roundabouts for circulating traffic to flow at similar speeds helps minimize the relative speeds between bicyclists and drivers, thereby improving safety and usability for bicyclists.

On multilane roundabouts, bicyclists are forced to change lanes to select the appropriate lane for their direction of travel. Vehicles are more likely to cut off bicyclists as they exit the roundabout. Therefore, a bicycle path that is separate and distinct from the circulatory roadway is preferred (e.g., a shared bicycle-pedestrian path of sufficient width and appropriately marked to accommodate bicyclists and pedestrians around the perimeter of the roundabout). See Section 8-6.0 for information on shared-use paths.

11. Illumination – Adequate lighting should be provided at roundabouts for two primary reasons. First, approaching drivers should be able to perceive the general layout of a roundabout in time to make the appropriate maneuvers. Second, the constrained curve radius of a roundabout may limit the effectiveness of headlights, making the roadway lighting system a key element for nighttime visibility of potential obstructions and other users (e.g., pedestrians).

6-6.0 INTERSECTION SIGHT DISTANCE

6-6.01 General

At each intersection, the potential exists for several different types of intersection conflicts. Therefore, Intersection Sight Distance (ISD) that exceeds SSD along the mainline should be provided at each intersection to reduce the possibility of these conflicts occurring. ISD consists of sight triangles that are clear of obstructions that could block the driver's view of other vehicles at or approaching an intersection. The two types of ISD sight triangles are:

- Departure Sight Triangles – Departure sight triangles provide sight distance to allow vehicles stopped on a crossing route to enter or cross the mainline. This type of sight triangle should be provided in each quadrant of an intersection approach that is controlled by a stop or a yield sign.

- Approach Sight Triangles – Approach sight triangles provide sight distance for a vehicle on the mainline to anticipate or avoid a potential collision with a vehicle that is entering or crossing the intersection from a crossing route. This type of sight triangle is typically used at channelized intersections where sight flares are provided. See Section 6-7.04 for more information on approach sight triangles.

The methods for determining sight distance at intersections are based upon the same principles as SSD, but with modified assumptions based on observed driver behavior. Less sight distance may be needed at roundabouts, signalized intersections, and all-way stop intersections. For more information on ISD for roundabouts, see NCHRP Report 672 *Roundabouts: An Informational Guide – Second Edition*.

The following subsections contain information for determining the dimensions of the departure sight triangle for undivided and divided roadways.

6-6.02 Basic Criteria

The driver of a vehicle stopped at an intersection should have a clear sight triangle to depart from the intersection and enter or cross the mainline. Departure sight triangles should be provided in each quadrant of a non-signalized intersection, but they should be considered for signalized intersections also. Figure 6-6-A shows departure sight triangles to the left and to the right for a vehicle stopped on a crossing route at an intersection with a 2-lane mainline.

The legs of the sight triangles should be determined as follows:

- Crossing route – The location of the driver's eye on the crossing route is typically assumed to be 15 feet from the edge of traveled way of the mainline and in the center of the lane on the crossing route. However, the leg of the sight triangle along the crossing route should also include the distance to the center of the approaching vehicle on the mainline. Therefore, the length of the sight triangle along the crossing route is the sum of the distance from the mainline (typically 15 feet) plus half of the lane width of the mainline for vehicles approaching from the left, and 1.5 times the lane width of the mainline for vehicles approaching from the right.

  If the mainline is a multilane roadway, the location of the vehicle approaching from the left is assumed to be the center of the outside travel lane, and the location of the vehicle approaching from the right is assumed to be the center of the closest travel lane.

  If the intersection is skewed, the lengths of the travel paths for some turning and crossing maneuvers will be increased. Therefore, the length of the leg along the crossing route should be increased by dividing the total widths of the lanes by the sine of the intersecting angle.

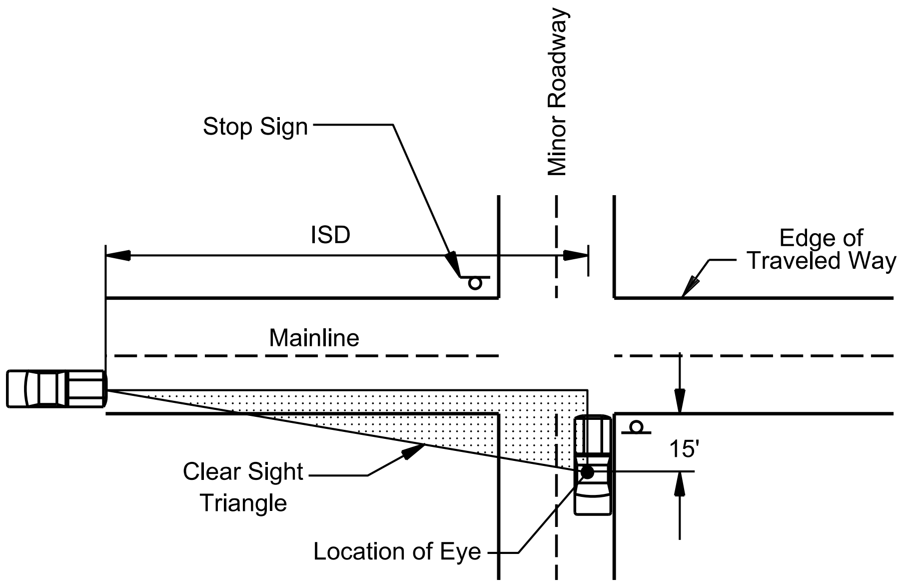
- Mainline – The Department uses gap acceptance time (gap time) as the conceptual basis for ISD criteria. The length of the sight triangle leg or ISD along the mainline is determined using the following equation:

  $$ISD = 1.467\, V_{major}\, t_g \qquad \text{(Equation 6-6-1)}$$

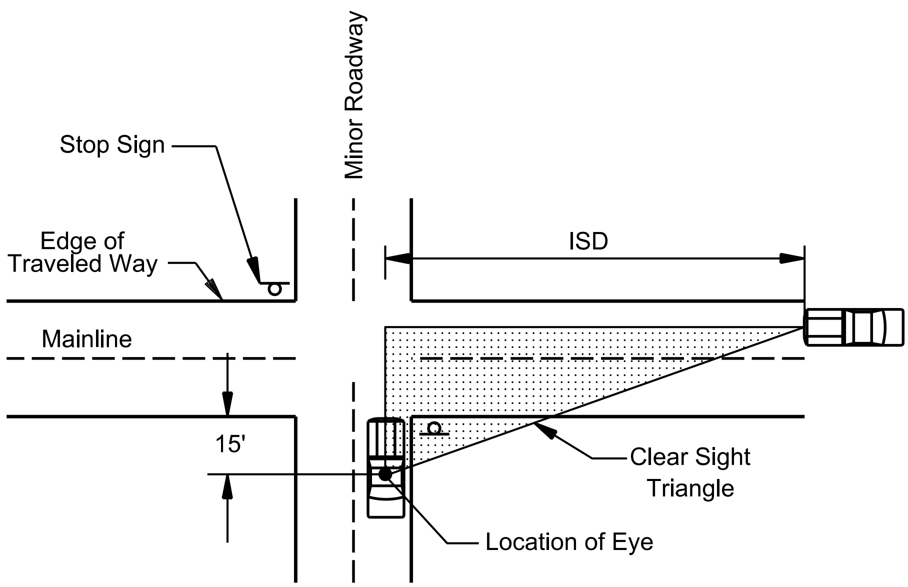
  Where:

  ISD = length of sight triangle leg along mainline (feet)
  V_{major} = design speed of mainline (miles per hour)
  t_g = gap time for entering the mainline (sec)

  The gap time varies according to the design vehicle, grade on the crossing route approach, number of lanes on the mainline, type of vehicular movement, and intersection skew.



(a) Clear Sight Triangle For Viewing
Traffic Approaching From The Left



(b) Clear Sight Triangle For Viewing
Traffic Approaching From The Right

CLEAR SIGHT TRIANGLES (STOP-CONTROLLED) INTERSECTIONS
Figure 6-6-A

Section 6-6.08 presents several examples on the application of ISD. Within the sight triangle, the driver's view should be clear of obstructions, such as buildings, bridge elements (e.g., bridge parapets, piers, abutments, bridge rail, guardrail), retaining walls, parked vehicles, existing ground line, vegetation, signs, fencing, etc. The horizontal and vertical alignments of each intersecting roadway, as well as the height and position of the object, determines whether an object may be considered as a sight obstruction. The driver's eye height is assumed to be 3.5 feet above the mainline roadway surface, and the object to be seen is assumed to be 3.5 feet above the surface of the intersecting crossing route. Section 6-7.04 provides more information on how to determine if an object is a sight obstruction.

For obstructions that are not feasible to be removed at existing intersections, the intersection should be considered for relocation, whenever feasible; however, other mitigation strategies may be more feasible at restricted locations.

6-6.03 Vehicles Entering Mainline

To determine the ISD for vehicles turning left or right onto the mainline, Equation 6-6-1 and gap time from Table 6-6-A should be used. Table 6-6-B, which solves Equation 6-6-1, provides the ISD values for all design vehicles on 2-lane, level roadways. The following information should also be used when determining the minimum ISD:

1. 2-Lane Roadways – There is minimal difference in the gap acceptance times between left- and right-turning drivers; therefore, only one gap time is provided.

2. Right Turns Onto Multilane Roadways – Because the turning vehicle is assumed to be turning into the nearest right through lane, no adjustment to the gap time is needed.

3. Left Turns Onto Multilane Roadways – For multilane roadways, the gap times presented in Table 6-6-A should be adjusted to account for the recommended distance for the turning vehicle to cross the additional lanes and/or median. The following apply:

    a. Undivided – For left turns onto multilane roadways, 0.5 seconds should be added for passenger cars or 0.7 seconds for trucks for each additional lane, in excess of one, to be crossed by the turning vehicle. The left-turning driver is assumed to enter the inside travel lane on the opposite side of the mainline. For example, the gap time for a passenger car turning left onto an undivided 5-lane roadway with a 12-foot CTWLTL would be 7.5 seconds, plus 0.5 seconds for each additional lane (inside travel lane and CTWLTL) to be crossed. The recommended adjusted gap time is therefore 8.5 seconds. Where striped gore areas are to be crossed instead of actual lanes, the gore width should be divided by 12 feet to determine the corresponding number of lanes.

    b. Divided With Narrow Median – For a multilane roadway that does not have a median wide enough to store a stopped vehicle, the median width should be divided by 12 feet to determine the corresponding number of lanes, and then the criteria in the preceding paragraph should be used to determine the total gap time.

Table 6-6-A
GAP TIMES
(Right or Left Turn from Crossing Route)

Design Vehicle	Gap Time (t_g) (sec)
Passenger Car	7.5
Single-Unit Truck	9.5
Tractor/Semitrailer	11.5

Table 6-6-B
2-LANE INTERSECTION SIGHT DISTANCES
(Right or Left Turn from Crossing Route)

Mainline Design Speed (V_{major}) (mph)	ISD (ft)		
	Passenger Cars	Single-Unit Trucks	Tractor/Semitrailers
20	225	280	340
25	280	350	425
30	335	420	510
35	390	490	595
40	445	560	675
45	500	630	760
50	555	700	845
55	610	770	930
60	665	840	1015
65	720	910	1100

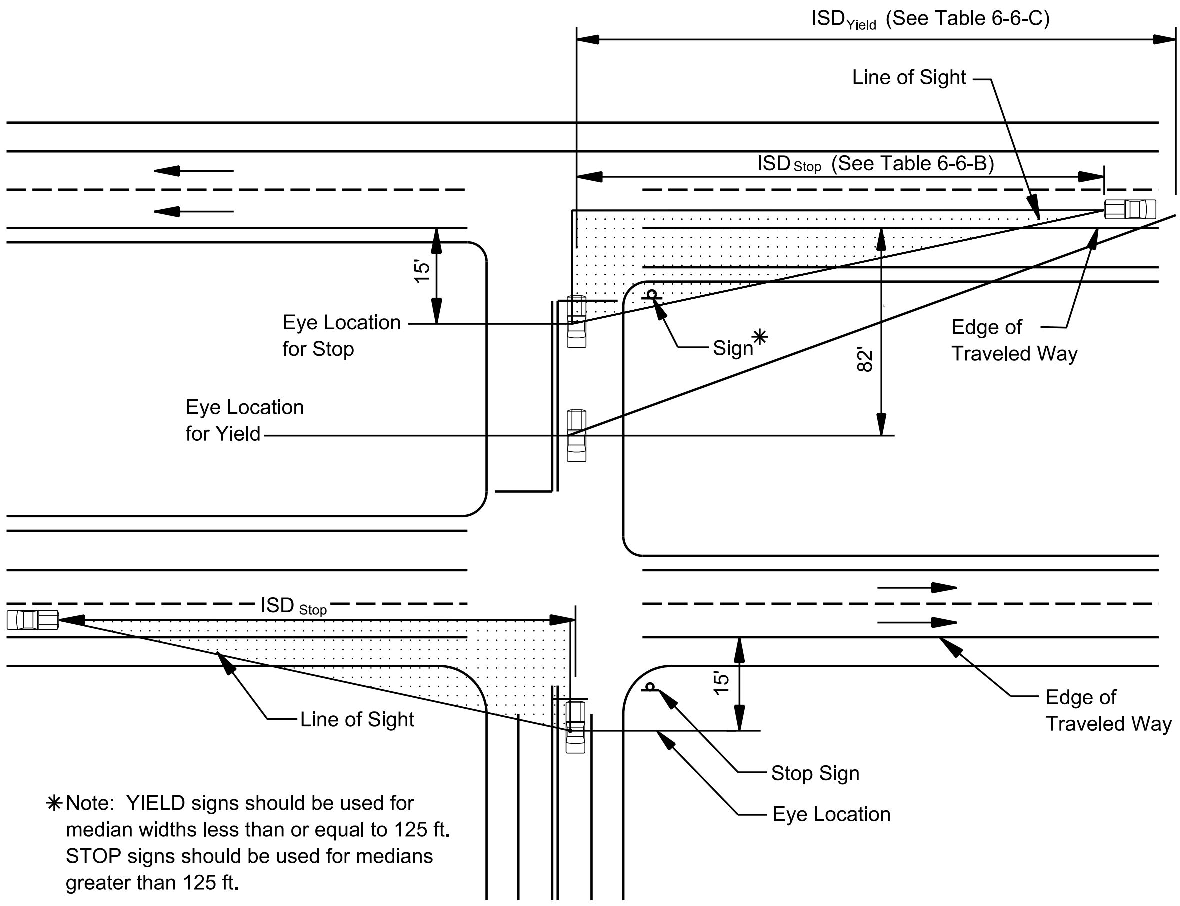
Note: These ISD values assume a crossing route storage platform grade is less than or equal to +3%.

c. Divided With Wide Median – For divided roadways with a median wide enough to store the design vehicle, the ISD should be evaluated in two steps as illustrated in Figure 6-6-B:

Step 1: (Initial Approach) – STOP signs should be used on the crossroad approach to the mainline. With the vehicle stopped on the crossing route (the bottom portion in Figure 6-6-B), the gap times and distances for a vehicle turning right (Tables 6-6-A and 6-6-B) should be used to determine the applicable ISD. Under some circumstances, the crossing maneuver should be checked to determine if it is the critical movement. Crossing criteria are discussed in Section 6-6.04.

Step 2: 125 feet or Less Median Width – On divided multilane roadways with a median width of 125 feet or less, YIELD signs should be used within the median for the crossing road. Table 6-6-C provides ISD criteria within the median for yield control. The 82 feet shown in Figure 6-6-B is the recommended distance for the driver to slow down from 20 miles per hour to 10 miles per hour to make the left turn without stopping.

Median Width Greater Than 125 feet – For median widths greater than 125 feet, STOP signs should be used on the crossroad within the median. With the vehicle stopped in the median (top portion in Figure 6-6-B), the designer should assume a 2-lane roadway and use the gap times and distances for vehicles turning left (Tables 6-6-A and 6-6-B) to determine the applicable ISD.



INTERSECTION SIGHT DISTANCE
(Divided Facilities)
Figure 6-6-B

Table 6-6-C
MEDIAN INTERSECTION SIGHT DISTANCES
(Yield-Controlled)

Mainline Design Speed (V_{major}) (mph)	ISD (ft)		
	Passenger Cars	Single-Unit Trucks	Tractor/Semitrailers
35	415	515	620
45	530	665	795
55	650	810	975
65	765	960	1150

Note: *These ISD values assume a crossing route storage platform grade is less than or equal to +3% and a 20 mile per hour design speed for the crossing route within the median.*

4. Approach Grades – If the storage platform grade on the crossing route is greater than +3%, the following adjustments should be made to the basic gap times in Table 6-6-A:

   a. Left Turns – The percent grade on the approach should be multiplied by 0.2 and added to the base gap time.

   b. Right Turns – The percent grade on the approach should be multiplied by 0.1 and added to the base gap time.

   The adjusted gap time (t_g) should be used in Equation 6-6-1 to determine the ISD.

5. Trucks – ISD should be provided for a single-unit truck at most intersections. At some intersections (e.g., near truck stops, interchange ramps, grain elevators, industrial parks), a tractor/semitrailer may be the more appropriate design vehicle for determining the ISD. The gap times for single-unit and tractor/semitrailer trucks are provided in Table 6-6-A. Truck ISD values for level, 2-lane roadways are presented in Table 6-6-B.

6. Eye/Object Height – The eye height for passenger cars is assumed to be 3.5 feet above the surface of the crossing route. The object height (approaching vehicle on the mainline) is also assumed to be 3.5 feet. An object height of 3.5 feet assumes that a sufficient portion of the oncoming vehicle should be visible to identify by the crossing route driver. If the number of trucks is sufficient to warrant consideration, an eye height of 7.6 feet should be used for tractors/semitrailers and six feet for single-unit trucks or buses. If a truck is the assumed entering vehicle, the object height is still 3.5 feet for a passenger car on the mainline.

7. Skew – At skewed intersections where the intersection angle is less than 75 degrees, adjustments may be recommended to account for the extra distance for the vehicle to travel across opposing lanes. For more guidance, see AASHTO's *A Policy on Geometric Design of Highways and Streets*.

8. Examples – Section 6-6.08 provides examples on the application of ISD.

6-6.04 Vehicle Crossing Straight Through

In the majority of cases, the ISD for turning vehicles should provide adequate sight distance to allow a vehicle to cross the mainline. However, crossing sight distance may be the more critical movement where:

- left and/or right turns are not permitted from a specific approach and the crossing maneuver is the only legal or expected movement (e.g., indirect left turns)
- the design vehicle crosses the equivalent width of more than six lanes
- a substantial volume of heavy vehicles crosses the roadway and there are steep grades on the crossing route approach

Equation 6-6-1 and the gap times and adjustment factors in Table 6-6-D should be used to determine the ISD for crossing maneuvers. Where medians are present, the median width should be included in the overall length to determine the adjusted gap time and this width divided by 12 feet to determine the corresponding number of lanes for the crossing maneuver.

Table 6-6-D
GAP TIMES
(Crossing Maneuvers)

Design Vehicle	Gap Time (t_g) (sec)
Passenger Car	6.5
Single-Unit Truck	8.5
Tractor/Semitrailer	10.5

Notes:

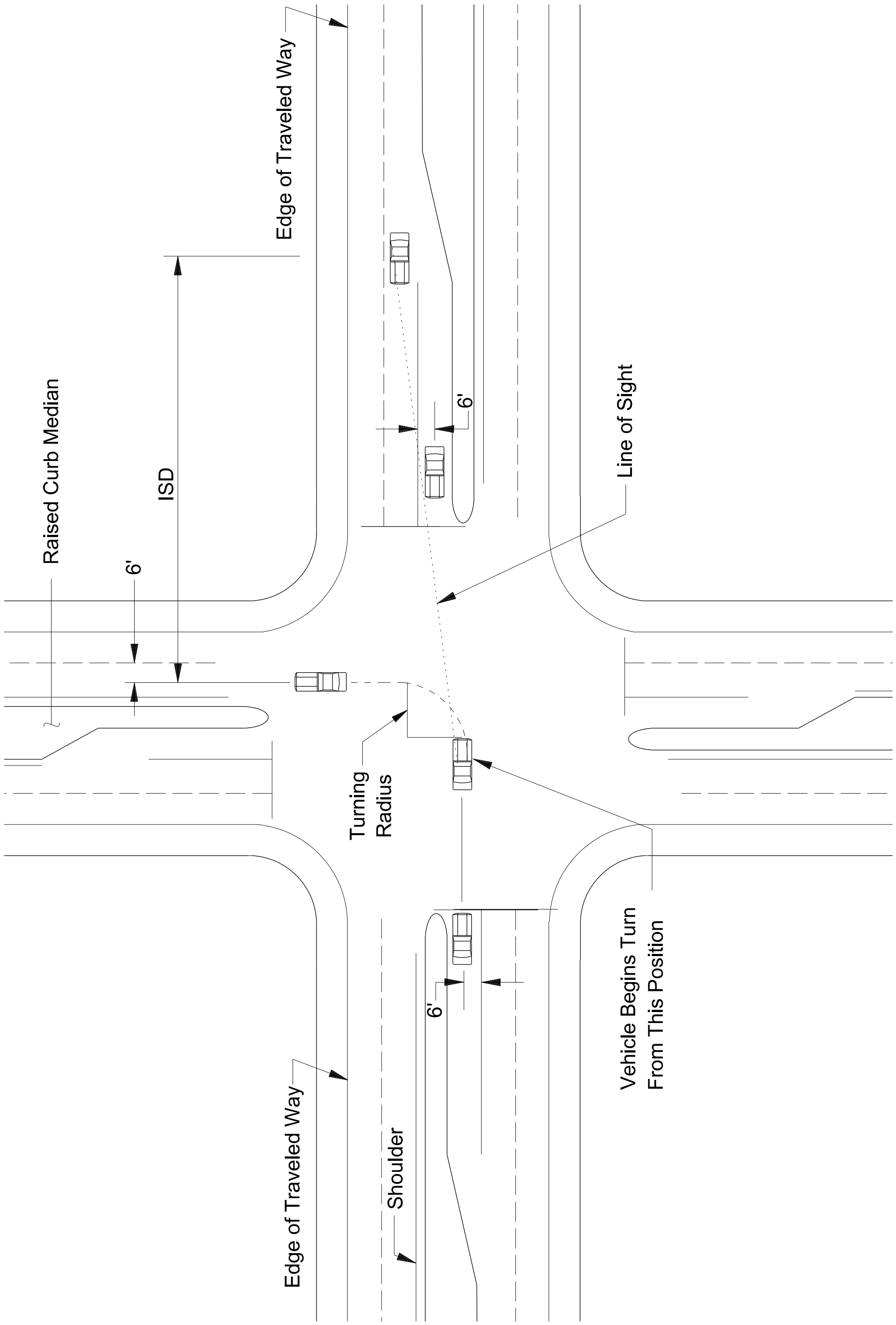
1. *Multilane Roadway – Where the design vehicle crosses a mainline with more than two lanes, 0.5 seconds should be added for passenger cars or 0.7 seconds for trucks for each additional lane in excess of two. See the discussion in Section 6-6.04 for additional information.*
2. *Approach Grade – If the storage platform grade on the crossing route is greater than +3%, the percent grade of the minor road approach should be multiplied by 0.2 and added to the base gap time.*

6-6.05 Stopped Vehicle Turning Left from Mainline

At intersections, not only should the recommended ISD for the crossing route be provided, but also the recommended sight distance for a stopped vehicle turning left from the mainline should be provided, as illustrated in Figure 6-6-C. A driver should be able to see straight ahead for a sufficient distance to turn left and clear the opposing travel lanes before an approaching vehicle reaches the intersection.

While the proposed profile for the mainline is controlled by the sight distance for the crossing route, the sight line for a vehicle turning left from the mainline could still be obstructed by opposing left-turning vehicles, bridge rails, guardrails, vertical or horizontal curves, landscaping, signs, or utilities. Therefore, intersections should be designed so that the sight line for left-turning vehicles is clear of such obstructions. For obstructions that are not feasible to be removed at existing intersections, the intersection should be considered for relocation, whenever feasible; however, other mitigation strategies may be more feasible at restricted locations.

Equation 6-6-1 and the gap times from Table 6-6-E should be used to determine the applicable ISD for the left-turning vehicle. Where the left-turning vehicle crosses more than one lane, 0.5 seconds should be added for passenger cars or 0.7 seconds for trucks for each additional lane in excess of one. Where medians are present, their effect should be considered in the same manner as discussed in Section 6-6.03. Table 6-6-F provides the ISD values for various design vehicles under two common left-turning situations.



Note: See Table 6-6-F for ISD values.

INTERSECTION SIGHT DISTANCE FOR A STOPPED VEHICLE TURNING LEFT FROM MAINLINE
Figure 6-6-C

Table 6-6-E
GAP TIMES
(Left-Turning Vehicles from Mainline)

Design Vehicle	Gap Time (t_g) (sec)
Passenger Car	5.5
Single-Unit Truck	6.5
Tractor/Semitrailer	7.5

Table 6-6-F
INTERSECTION SIGHT DISTANCES
(Left-Turning Vehicles from Mainline)

Mainline Design Speed (V_{major}) (mph)	ISD (ft)					
	Passenger Cars		Single-Unit Trucks		Tractor/Semitrailers	
	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes
20	165	180	195	215	220	240
25	205	220	240	265	275	300
30	245	265	290	320	330	360
35	285	310	335	370	385	425
40	325	335	385	425	440	485
45	365	400	430	480	495	545
50	405	440	480	530	550	605
55	445	485	525	585	605	665
60	485	530	575	635	660	725
65	525	575	620	690	715	785

6-6.06 All-Way Stop

At intersections with all-way stop control, ISD should preferably be provided for each approach. At a minimum, sufficient sight distance should be provided so that the first stopped vehicle on each approach is visible to all other approaches.

6-6.07 Signal-Controlled Intersections

At signalized intersections, sufficient sight distance should be provided so that the first vehicle on each approach is visible to all other approaches. Because right-turn-on-red is generally allowed, the ISD for a stop-controlled, right-turning vehicle, as presented in Section 6-6.03, should be provided to the left. Otherwise, the right-turn-on-red movement may necessitate restriction. In addition, if the traffic signal is to be placed on 2-way flash operation during off-peak or nighttime conditions, the ISD criteria should be provided, as discussed in Section 6-6.03, for a stop-controlled intersection.

6-6.08 Examples of ISD Applications

The following examples illustrate the application of the ISD criteria.

Example 6-6-1

Given: Crossing route intersects a 5-lane roadway that includes a CTWLTL.
Crossing route is a stop-controlled roadway.
Design speed of the mainline is 50 miles per hour.
All travel lane widths are 12 feet.
The CTWLTL width is 14 feet.
Trucks are not a factor.

Problem: Determine the ISD to the left and to the right from the crossing route.

Solution: The following steps apply:

Step 1: For the vehicle turning right, the ISD to the left can be determined directly from Table 6-6-B. For the 50 mile per hour design speed, the ISD to the left is 555 feet.

Step 2: For the vehicle turning left, the ISD should reflect the additional time to cross the additional lanes as discussed in Section 6-6.03.

a. First, the extra width of the one additional travel lane and the CTWLTL should be determined and then this number should be divided by 12 feet:

$$\frac{(12 + 14)}{12} = 2.2 \text{ lanes}$$

b. Next, the number of lanes determined above should be multiplied by 0.5 seconds to determine the additional time:

(2.2 lanes)(0.5 sec/lane) = 1.1 seconds

c. The additional time should be added to the basic gap time of 7.5 seconds and inserted into Equation 6-6-1:

$$S = (1.467)(50)(7.5 + 1.1) = 631 \text{ feet}$$

A minimum ISD of 631 feet should be provided to the right for the left-turning vehicle.

Step 3: The crossing vehicle, as discussed in Section 6-6.04, should be checked.

a. First, the extra width of the two additional travel lanes and the CTWLTL should be determined and then this number should be divided by 12 feet:

$$\frac{(12 + 12 + 14)}{12} = 3.2 \text{ lanes}$$

b. Next, the number of lanes from above should be multiplied by 0.5 seconds to determine the additional time:

(3.2 lanes)(0.5 sec/lane) = 1.6 seconds

c. The additional time should be added to the basic gap time of 6.5 seconds and this value inserted into Equation 6-6-1:

S = (1.467)(50)(6.5 + 1.6) = 594 feet

The 594 feet for the crossing maneuver is less than the 631 feet recommended for the left-turning vehicle and, therefore, is not the critical maneuver.

Example 6-6-2

Given: Crossing route intersects a 4-lane divided roadway.
Crossing route is a stop-controlled roadway.
Design speed of the mainline is 65 miles per hour.
All travel lane widths are 12 feet.
The median width is 125 feet.
Trucks are not a factor.

Problem: Determine the ISD to the left and to the right from the crossing route.

Solution: The following steps apply:

Step 1: For the vehicle turning right, the ISD to the left can be determined directly from Table 6-6-B. For the 65 mile per hour design speed, the ISD to the left is 720 feet.

Step 2: The crossing maneuver should be determined to see if it is critical. See Section 6-6.04. No adjustments should be made to the base time of 6.5 seconds because the median is wide enough to store a stopped vehicle. Therefore, Equation 6-6-1 should be used directly:

ISD = (1.467)(65)(6.5) = 620 feet

The crossing maneuver is less than the right-turning maneuver and, therefore, is not critical.

Step 3: For the 125-foot median, YIELD signs should be used within the median on the crossroad. For the vehicle turning left, the designer may assume that the passenger car will not stop in the median. See Figure 6-6-B. The ISD to the right can be determined directly from Table 6-6-C. For the 65 mile per hour design speed, the ISD to the left is 765 feet. The stopped and crossing maneuvers will not be critical.

Example 6-6-3

Given: Crossing route intersects a 2-lane roadway.
Crossing route is a stop-controlled roadway.
Design speed of the mainline is 60 miles per hour.
All travel lane widths are 12 feet.
The storage platform grade on the crossing route is 4.5%.
Tractor/semitrailer trucks are a factor.

Problem: Determine the ISD to the left and to the right from the crossing route.

Solution: The following steps apply:

Step 1: For the left-turning vehicle, the base gap time from Table 6-6-A is 11.5 seconds. The additional time due to the approach grade (0.2 seconds per percent grade) should be added to the base gap time as discussed in Section 6-6.03:

$$(0.2)(4.5) + 11.5 = 12.4 \text{ seconds}$$

Then, using Equation 6-7-1:

$$ISD = (1.467)(60)(12.4) = 1091 \text{ feet}$$

Step 2: The ISD for the right-turning vehicle is determined similarly:

$$(0.1)(4.5) + 11.5 = 12 \text{ seconds}$$

Then, using Equation 6-7-1:

$$ISD = (1.467)(60)(12.0) = 1056 \text{ feet}$$

Step 3: The crossing maneuver will not be critical.

6-7.0 CHANNELIZATION

6-7.01 Type

Channelization of at-grade intersections is the method by which traffic entering an intersection is directed into definite paths by islands. Most at-grade intersections with low-volume roadways should be simple, non-channelized intersections. As the significance of the intersecting roadway increases, various channelization treatments should be considered. Table 6-7-A presents general criteria that should be used to make an initial determination on channelization. Each intersection should be evaluated to determine the applicability of the criteria in Table 6-7-A.

At intersections where the approaching traffic volumes are approximately equal and a signal is not warranted, all-way stop control may be warranted. The *MUTCD* presents minimum traffic volume warrants for all-way stop control intersections.

Additionally, the methodology in the *Highway Capacity Manual* should be used to determine if the all-way stop can accommodate the intersection traffic volumes at the desired LOS. All-way stops should not be proposed where either roadway is a divided facility.

Table 6-7-A
CHANNELIZATION WARRANTS

Current ADT (Crossroad)	Suggested Channelization Treatment	
	Crossroad	**Mainline**
Under 300	No channelization	Left-turn lanes on 4-lane divided only
300 – 1000	Simple channelization	Left-turn lanes
*1000 – 2000	Channelize with left-turn lanes	Left-turn lanes and right-turn lanes
Over 2000	Signal warrant and capacity analyses should be conducted.	

**Turning movement count from Planning Division is recommended at all intersections where crossroad ADT is 1000 or more. Turning movement count may be requested at intersections where crossroad ADT is between 300 and 1000 to determine which movements should be channelized.*

The following subsections provide information on the design of channelized intersections, and Figure 6-7-A provides a typical layout of a rural channelized intersection.

6-7.02 Design of Islands

The design of traffic islands should consider site-specific functions, including definition of vehicular paths, separation of traffic movements, prohibition of movements, protection of pedestrians, and placement of traffic control devices. Islands can be grouped into the following functional categories:

Directional Islands – Control and direct traffic movements and guide the driver into the proper channel.

Divisional Islands – Separate opposing traffic flows, alert the driver to the crossroad ahead, and regulate traffic through the intersection. These islands are often introduced at intersections on undivided roadways and are particularly advantageous in controlling left turns at skewed intersections and preventing wrong-way turns into right-turning traffic lanes.

Refuge Islands – Aid or protect pedestrians crossing a wide roadway.

The criteria in the following subsections should be considered when designing islands. The *MUTCD* and *PROWAG* should be referenced for additional information.

6-7.02.01 Types of Islands

Traffic islands may be flush (striped) or raised, paved or turf, and triangular or elongated. Typically, the Department uses triangular-shaped raised islands formed by curbs, which define the boundary between the traveled way and the island more clearly than flush islands. Crossing movements are also improved for pedestrians at raised islands by providing refuge such that they can cross the intersection in two stages, and pedestrians with disabilities have better guidance about the location of a raised island than of a striped island.

The *Standard Drawings* include two types of curb and gutter with sloping curb faces for forming the edges of raised islands — Type 2 and Type 3. Type 2 curb and gutter, which is four inches in height and is on a plane slope, should be used at most intersections. Type 3 curb and gutter, which has a round-faced curb with a height of six inches, should typically only be used at urban intersections on low-speed roadways ($V \leq 45$ miles per hour) in conjunction with SS-2 inlets for drainage purposes. Flush islands should only be used in low-speed urban areas when the minimum island size cannot be provided (see Section 6-7.02.02) and where pedestrians are not anticipated.

Section 14-2.06.01 and the *Standard Drawings* provide more information on the different types of curb and gutter.

6-7.02.02 Minimum Size

Traffic islands should be large enough to command the driver's attention. Island shapes and sizes vary from one intersection to another. For rural intersections, the island width should desirably be a minimum of 25 feet as shown in Figure 6-7-A. Urban triangular islands should not be less than 12 feet on a side after the rounding of corners, but desirably should not be less than 15 feet. See Figure 6-7-B.

Elongated or divisional islands should not be less than four feet wide and 20 feet long. In restricted areas, an elongated island may be reduced to a minimum width of two feet. If a sign is expected to be located in the island, the offset between the face of curb and the edge of the sign should be a minimum of two feet. The 2-foot offset on each side of the sign and the width of the sign may control the minimum width of the island. The Traffic Engineering Division should be consulted to determine the width of proposed signs to be located in the island. Divisional islands on high-speed roadways ($V \geq 50$ miles per hour) should be a minimum of 100 feet in length.

Traffic islands that are intended to also provide pedestrian refuge shall be sized and positioned to provide a clear pedestrian path that has a minimum width of six feet and a minimum length of six feet. The path shall be clear of all obstructions (e.g., curbs, poles, sign posts, utility boxes).

6-7.02.03 Approach Treatment

Approach and trailing corners of curbed islands should desirably be constructed with 2-foot radii measured to the face of curb. The corner radius should desirably be five feet to the face of curb as shown in Figure 6-7-B. For good delineation, pavement markings or raised pavement markers should be placed in advance of the island approach to warn the driver. The *Standard Drawings* should be referenced for the appropriate pavement marking details.

6-7.02.04 Offsets

Traffic islands should be offset from the edge of the traveled way. The offset for rural islands should typically be the shoulder width. See Figure 6-7-B for offset guidance at urban islands. The offset distance should be measured from the edge of the traveled way to the face of curb. No additional offsets are recommended for auxiliary lanes.

6-7.02.05 Typical Pavement Markings

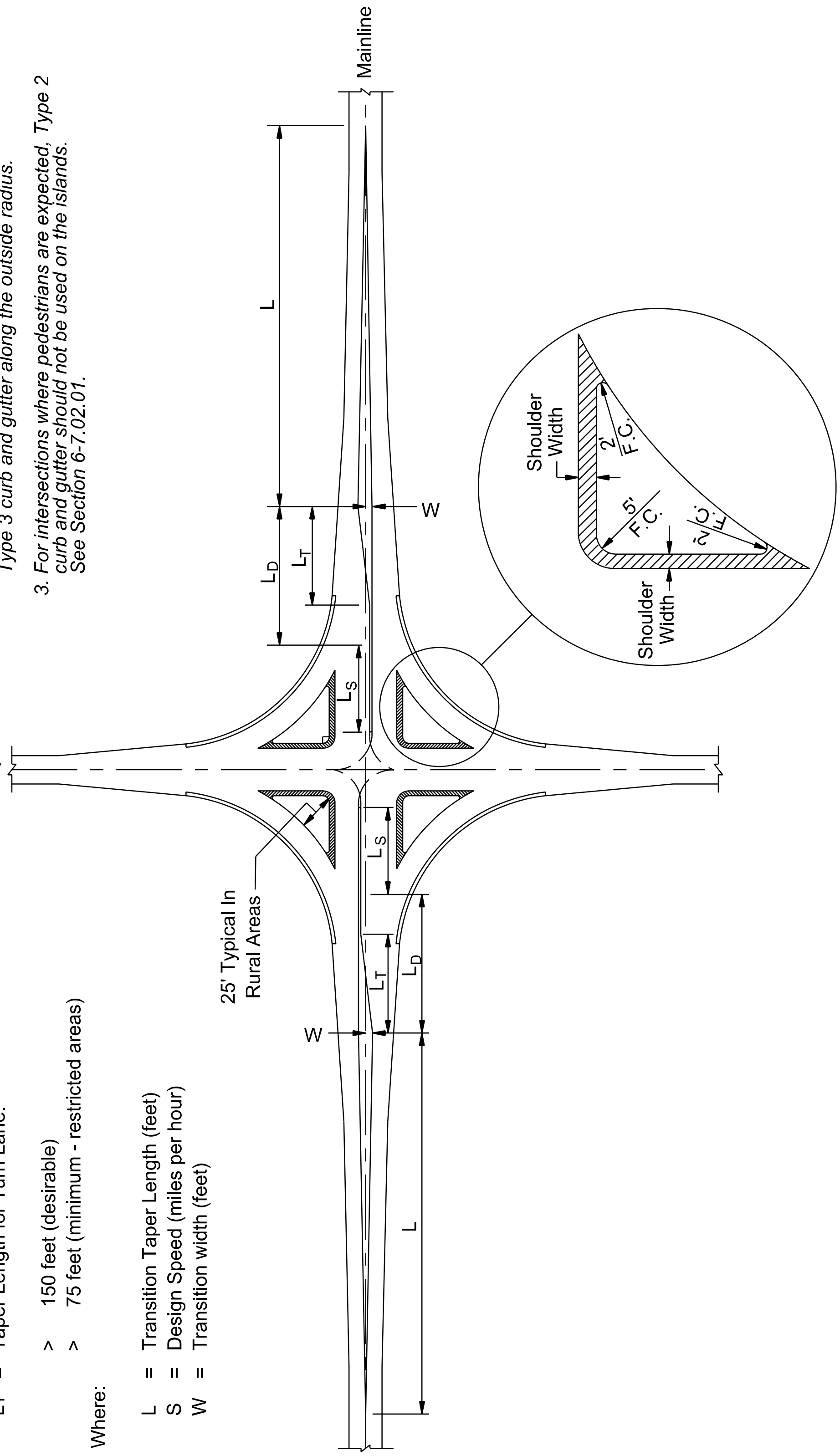
Curbed islands may be difficult to see at night because of the glare from the headlights of oncoming vehicles or from the lighting of roadside businesses. Therefore, curbed islands should have appropriate delineation with retroreflective pavement markings. See the *Standard Drawings* for typical pavement markings at channelized intersections.

Notes:

1. All curb and gutter should typically be Type 2 (low profile).

2. For urban intersections, drainage may dictate the use of Type 3 curb and gutter along the outside radius.

3. For intersections where pedestrians are expected, Type 2 curb and gutter should not be used on the islands. See Section 6-7.02.01.



Key: See Section 6-4.03

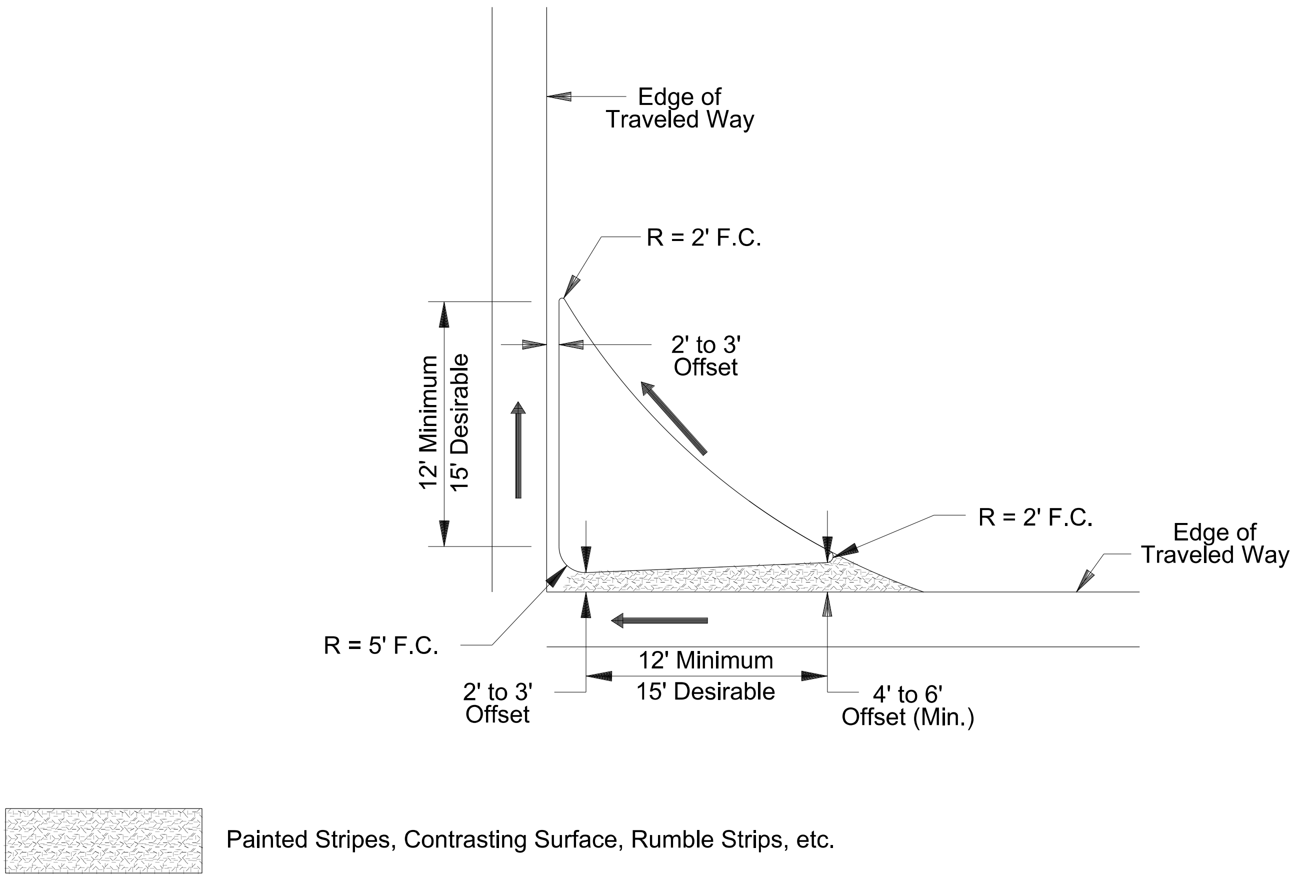
L_S = Storage Length (feet)
L_D = Deceleration Length (feet)
L_T = Taper Length for Turn Lane:

> 150 feet (desirable)
> 75 feet (minimum - restricted areas)

Where:

L = Transition Taper Length (feet)
S = Design Speed (miles per hour)
W = Transition width (feet)

TYPICAL LAYOUT (Channelized Intersection)
Figure 6-7-A



DESIGN OF TRIANGULAR ISLANDS
(Curbed Streets — Narrow Shoulders)
Figure 6-7-B

6-7.03 Turning Roadways

Turning roadways are channelized areas (separated by an island) for right-turning vehicles that are generally designed for lower speeds than through travel lanes. Turning roadways include at-grade intersections and controlled-terminal interchange ramps (See Chapter 7 for more information on interchange ramp terminals). The following subsections provide information on the design of turning roadways that takes all types of users (e.g., design vehicle, pedestrians) into consideration. If pedestrians are present or anticipated in the future, the turning roadway shall be designed such that all pedestrian facilities meet or exceed *ADA* requirements. The *PROWAG* should be referenced for additional information.

Turning roadway design does not apply to non-channelized intersections or free-flowing interchange ramps.

6-7.03.01 Turning Radius

The radii of a turning roadway should be coordinated with the controlling geometry of the intersection and the different types of users expected at the intersection. The radii selected should result in an island size that meets or exceeds the minimum island size as discussed in Section 6-7.02.02, but also provides a balance between the design vehicle and pedestrians, if present. On roadways in urban and suburban areas, the turning radius has a direct effect on the length of crosswalks and the amount of superelevation that pedestrians will traverse.

Figure 6-7-C illustrates a typical example of a turning roadway for an at-grade intersection using a simple curve with a taper offset. In the figure, as the angle of intersection (θ) decreases, the turning radius (R_1) increases and the turning radius (R_2) decreases.

6-7.03.02 Deceleration and Acceleration Lanes

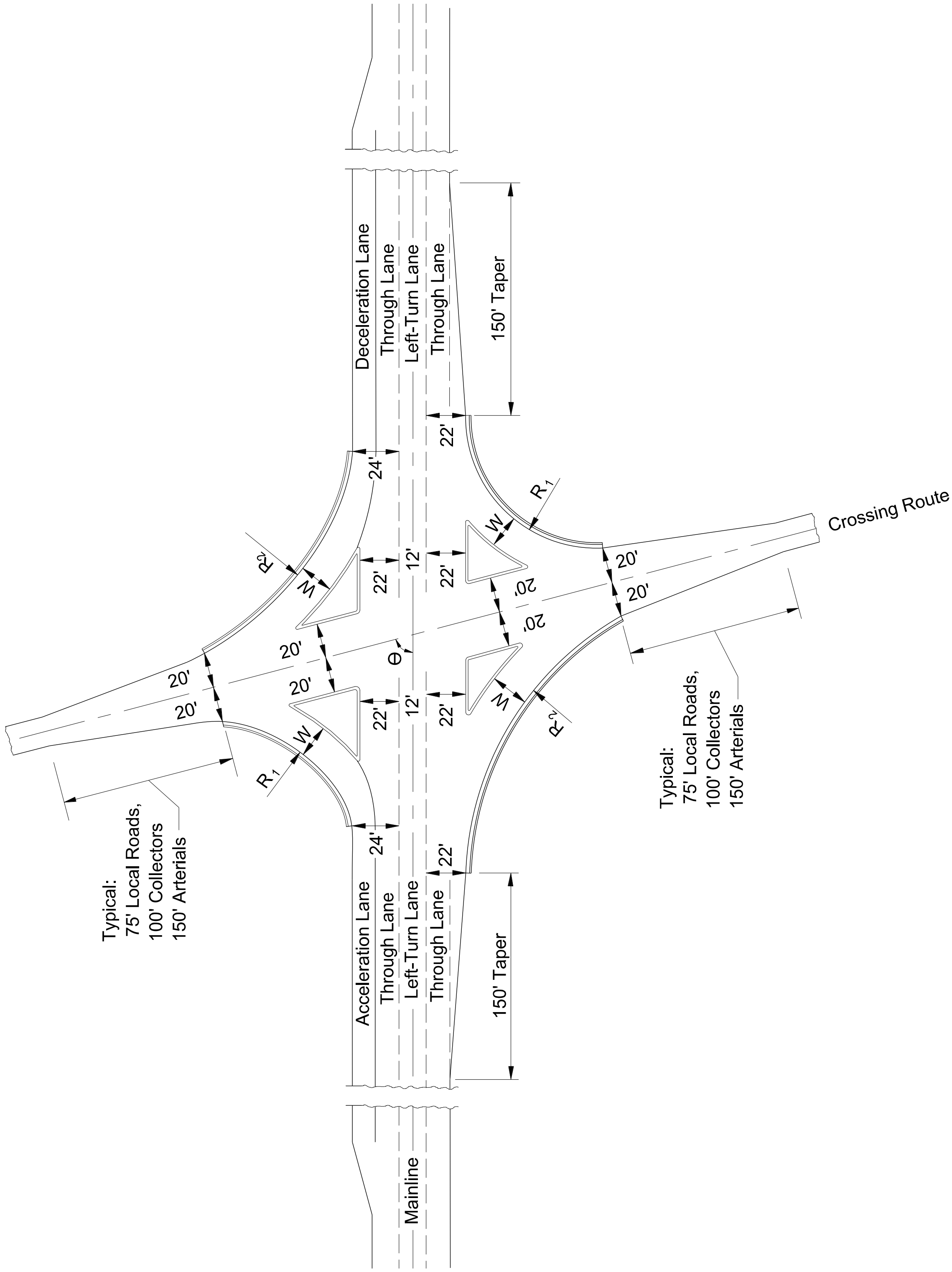
A deceleration or acceleration lane for a turning roadway should provide sufficient length for a driver to comfortably decelerate or accelerate without interference to the through traffic of the intersection.

Deceleration lanes provide the driver with a length of roadway to diverge from the traffic flow and decelerate to a lower design speed before entering the turning roadway. Table 6-7-B provides thresholds for deceleration lanes at turning roadways based on traffic volumes. If the combination of mainline volumes and turning volumes exceeds these criteria during the design hour, a deceleration lane should be analyzed.

Acceleration lanes provide a driver with a length of roadway to accelerate to the design speed of the mainline and safely merge into the traffic flow. Typically, acceleration lanes are not provided at non-channelized intersections. At channelized intersections, the use of acceleration lanes should be determined on a case-by-case basis considering traffic volumes, mainline speed, grades, sight distance, urban/rural location, type of traffic control, etc.

Recommended lengths of deceleration and acceleration lanes for turning roadways are provided in Table 6-7-C. However, these recommended lengths may warrant adjustment if there is a significant amount of trucks.

The width of a deceleration or acceleration lane should desirably be equal to the width of the adjacent through lane; however, the width should not be less than 10 feet.



Note: Where $\theta = 90°$, $R_1 = R_2 = 115$ ft to 150 ft
W = Width of turning roadway. See Section 6-7.03.04.

TURNING ROADWAY INTERSECTIONS
Figure 6-7-C

Table 6-7-B
THRESHOLDS FOR DECELERATION LANES AT TURNING ROADWAYS

Total Volume in One Direction (vph)	Right-Turning Volume (vph)
< 200	(Not Warranted)
200 – 300	120
300 – 400	100
400 – 500	85
500 – 600	70
> 600	50

Example 6-7-1

Given: 1-Way Volume = 350 vehicles per hour
Right Turns = 105 vehicles per hour

Problem: Determine if a deceleration lane to the turning roadway should be provided.

Solution: According to Table 6-7-B, the right-turning volume threshold is 100 vehicles per hour where the 1-way volume is 350 per hour. Therefore, with a turning volume of 105 vehicles per hour, a deceleration lane to the turning roadway may be warranted.

6-7.03.03 Cross-Slope Rollover

The cross-slope rollover of a turning roadway is the algebraic difference between the cross slope of the through lane and the cross slope of the turning roadway, including the tapers and, if present, acceleration/deceleration lanes. The maximum cross-slope rollover for various design speeds is provided in Table 6-7-D.

Abrupt changes in cross slope can adversely affect the stability of trucks and other vehicles with high centers of gravity. Therefore, form grades should be calculated to ensure comfortable superelevation transitions throughout the turning roadway. Form grade development can often be an iterative process, so modifications may be required to provide the desired geometric attributes and ensure proper drainage.

6-7.03.04 Width of Roadway

Widths of turning roadways should be determined primarily by the selected design vehicle and the radius of the controlling curve. Turning roadways should have sufficient width to allow the wheel tracks of the design vehicle to be within the edges of the traveled way by approximately two feet on each side. Table 6-7-E provides the recommended design widths of pavement for turning roadways. In urban and suburban areas, however, pedestrian usage should also be considered in determining the width of the turning roadway.

Table 6-7-C
RECOMMENDED LENGTHS OF ACCELERATION OR DECELERATION LANES FOR TURNING ROADWAYS

Design Speed of Mainline (mph)	Design Speed of Turning Roadway (mph)						
	Stop Condition	15	20	25	30	35	40
Deceleration Length (ft)							
30	235	200	170	140			
40	320	295	265	235	185	155	
50	435	405	385	355	315	285	225
55	480	445	440	410	380	350	285
60	530	500	480	460	430	405	350
65	570	540	520	500	470	440	390
Acceleration Length (ft)							
30	180	140					
40	360	300	270	210	120		
50	720	660	610	550	450	350	130
60	1200	1140	1100	1020	910	800	550
65	1410	1350	1310	1220	1120	1000	770

Notes:

1. *The deceleration or acceleration occurs within both the taper and the full-width lane up to the PC or PCC of the controlling curve (i.e., the lengths in the table include the taper length).*

2. *The minimum taper length for deceleration lanes is 150 feet. In restricted urban areas, the taper length may be reduced where a 150-foot length is not feasible, but should not be less than 75 feet.*

3. *The minimum taper length for acceleration lanes should be 150 feet for design speeds less than or equal to 40 miles per hour, and 300 feet for design speeds greater than 40 miles per hour.*

Table 6-7-D
MAXIMUM CROSS-SLOPE ROLLOVER AT TURNING ROADWAYS

Design Speed of Turning Roadway (mph)	Maximum Algebraic Difference in Cross Slope (Percent)
≤ 20	8
25 – 30	6
≥ 35	5

Table 6-7-E
RECOMMENDED DESIGN WIDTHS OF PAVEMENT FOR TURNING ROADWAYS

Radius on Inner Edge of Pavement R (ft)	Recommended Design Widths (ft)				
	P	**SU-40**	**S-BUS-40**	**WB-62**	**WB-67**
50	13	21	18	*	*
75	13	18	17	*	*
100	13	17	16	*	*
150	12	16	15	22	23
200	12	16	15	20	21
300	12	15	15	18	19
400	12	15	15	17	18
500	12	15	14	17	17
Tangent	12	14	14	15	15

** Larger radius recommended.*

Note: Tables 6-2-A and 6-2-B provide guidance on the applicable design vehicle that should be used for design.

6-7.04 <u>Sight Flare Criteria</u>

6-7.04.01 General

Section 6-6.0 introduced two different types of sight triangles for providing ISD at at-grade intersections – departure sight triangles and approach sight triangles. In addition to the ISD criteria based on gap acceptance time that is used to determine departure sight triangles, the driver on the mainline should have sufficient sight distance to see vehicles approaching the intersection on a crossing route and avoid a collision if the crossing route vehicle violates the traffic control. This type of sight triangle is called an approach sight triangle.

The Department typically uses approach sight triangles for intersections where the DHV of the mainline exceeds 300 and the current ADT of the intersecting crossing route exceeds 300. The resulting sight line between the approaching vehicles of the two roadways is called a sight flare. Figure 6-7-D illustrates the sight flare criteria for undivided and divided roadways.

The sight flare criteria will determine a sight triangle that should be free of all sight obstructions, such as, but not limited to, buildings, signs, driveways, large trees, utilities, and embankment. The legs of the sight triangle should be based on SSD rather than ISD, and should be measured as shown in Figure 6-7-D.

As part of the design of a sight flare, the elevation of the ground along the sight line should be checked to ensure the ground is not obstructing the line of sight. If it is, the ground line should be excavated to a minimum of one foot below the line of sight and additional right of way acquired for the backslope. Figure 6-7-E illustrates a sight flare resulting in additional right of way for a cut section.

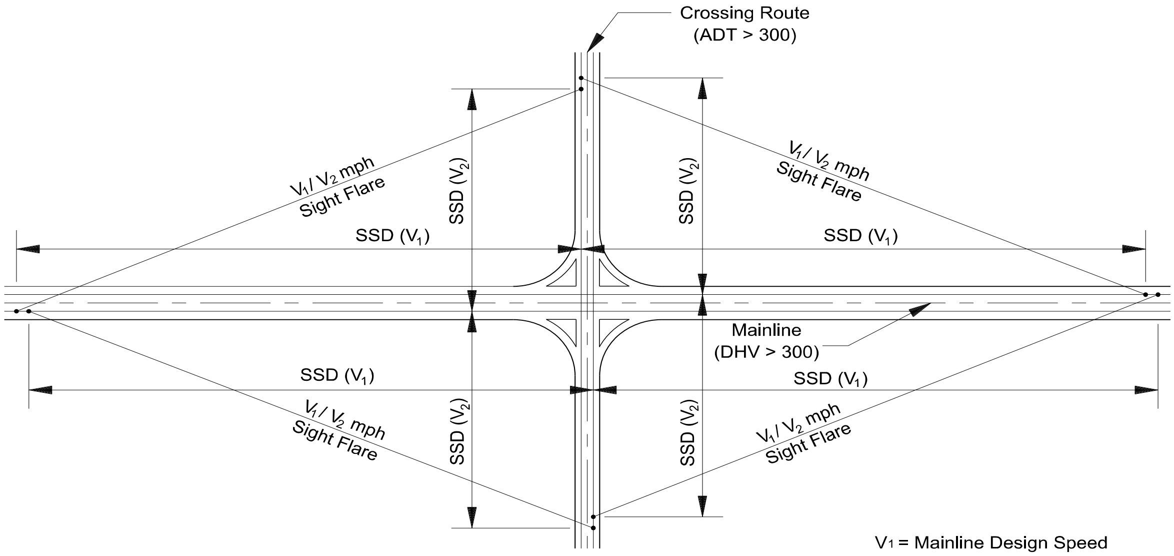
Figure 6-7-F provides cross sections for the example in Figure 6-7-E. To determine the maximum allowable elevation along the sight line, the ratio of the distances to the ratio of the elevation changes should be equated. An example calculation is provided in Figure 6-7-E. The table in Figure 6-7-E shows the results of the calculations for one quadrant of the example intersection. The cross sections in Figure 6-7-F show the sight line and cut for these elevations. No Access Right of Way should be shown as in Figure 6-7-G to avoid the future introduction of sight obstructions.

6-7.04.02 Existing Intersections

At existing intersections, a sight flare is desirable but is often not feasible due to existing development, especially in urban areas. In such cases, it is not necessary to meet the minimum sight flare criteria as shown in Figure 6-7-D. The following information should be applied:

- Development – Where development encroaches on a proposed minimum sight flare, a reduced sight flare with No Access Right of Way that avoids the development may be considered. If a reduced sight flare does not avoid the development, the sight flare may be eliminated.

- No Development – A sight flare should be shown on the plans for any quadrants that do not contain development, even in urban areas, to prohibit any future sight obstructions.



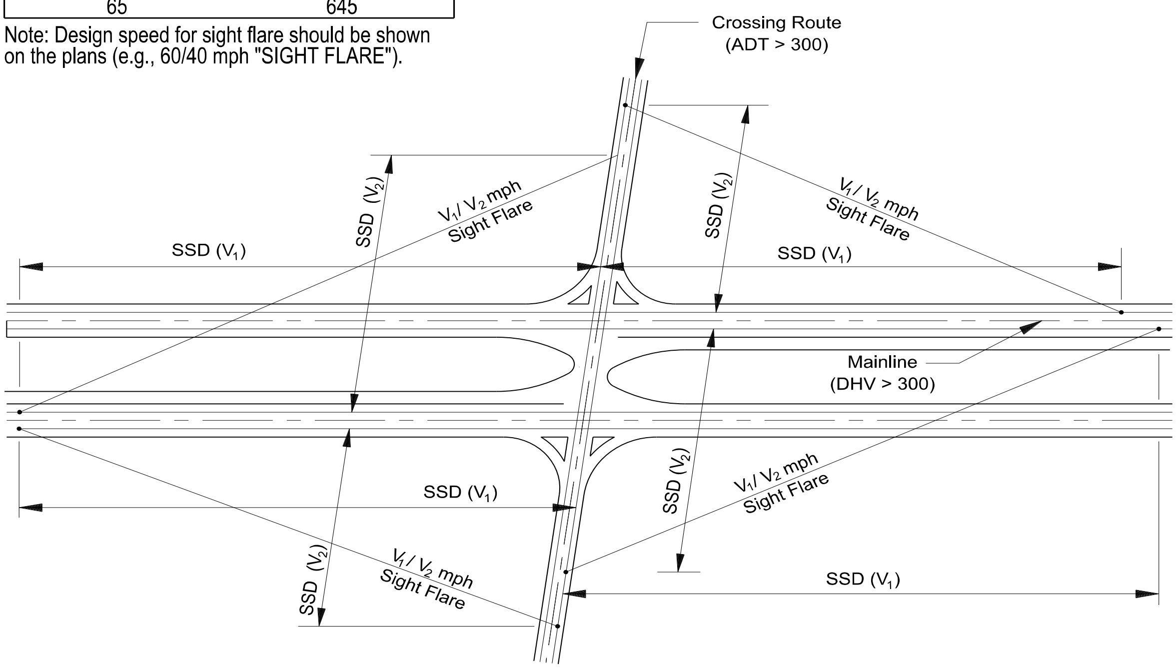
Design Speed (mph)	SSD (ft)
30	200
35	250
40	305
45	360
50	425
55	495
60	570
65	645

Note: Design speed for sight flare should be shown on the plans (e.g., 60/40 mph "SIGHT FLARE").

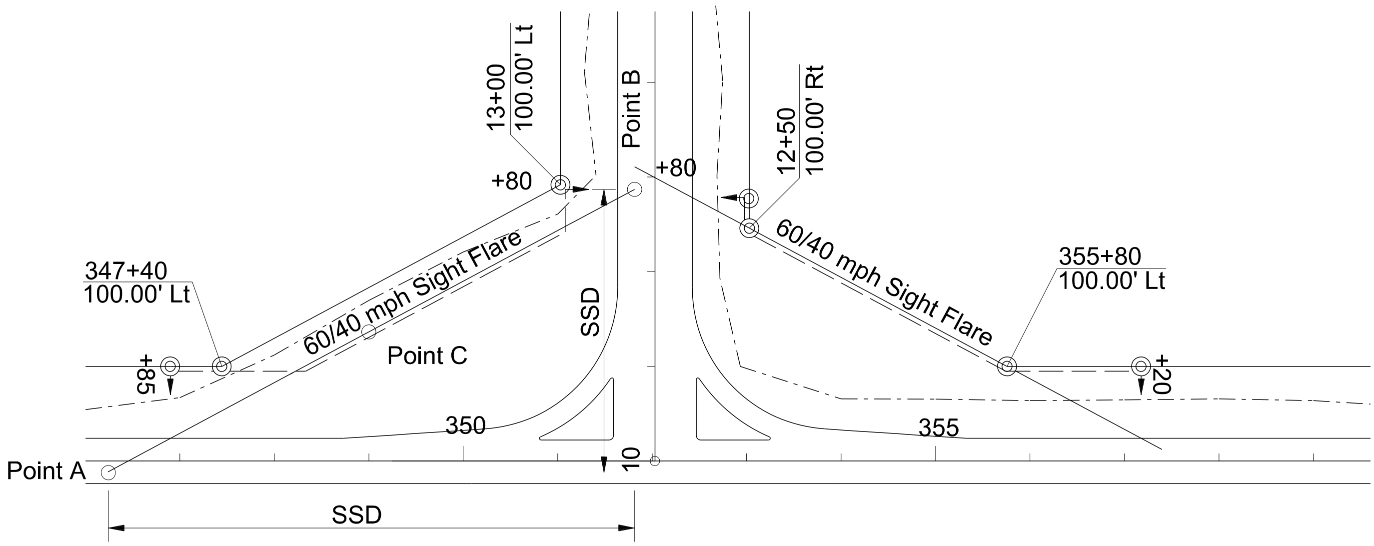
RECOMMENDED DESIGN SPEED V_2

4-LEGGED INTERSECTIONS:
Arterials = 50 mph
Collectors = 40 mph
Local Roads = 30 mph

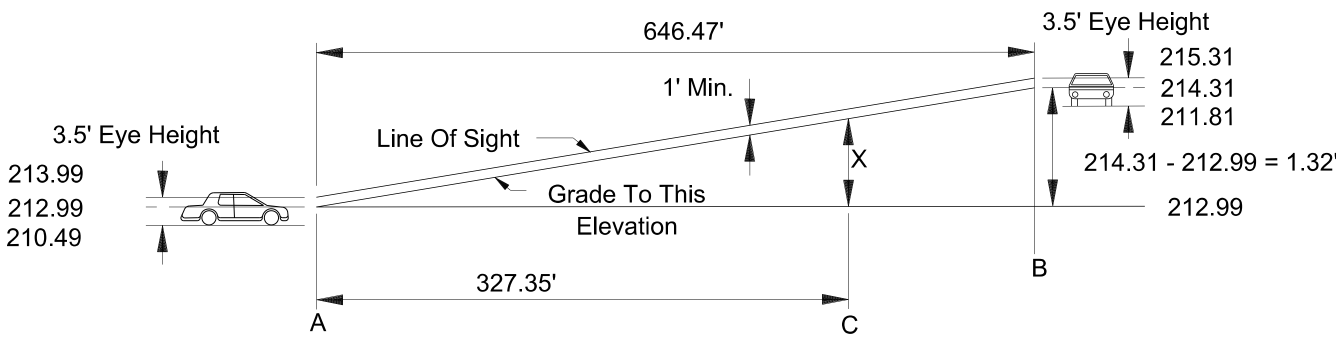
ALL T-INTERSECTIONS = 30 mph
(regardless of classification)



SIGHT FLARE CRITERIA AT INTERSECTIONS
Figure 6-7-D



Calculations for Station 349+00:



$$\frac{\text{Elev. Difference Between A \& C}}{\text{Elev. Difference Between A \& B}} = \frac{\text{Dist. From A To C}}{\text{Dist. From A To B}}$$

$$\frac{X}{214.31 - 212.99} = \frac{327.35}{646.47}$$

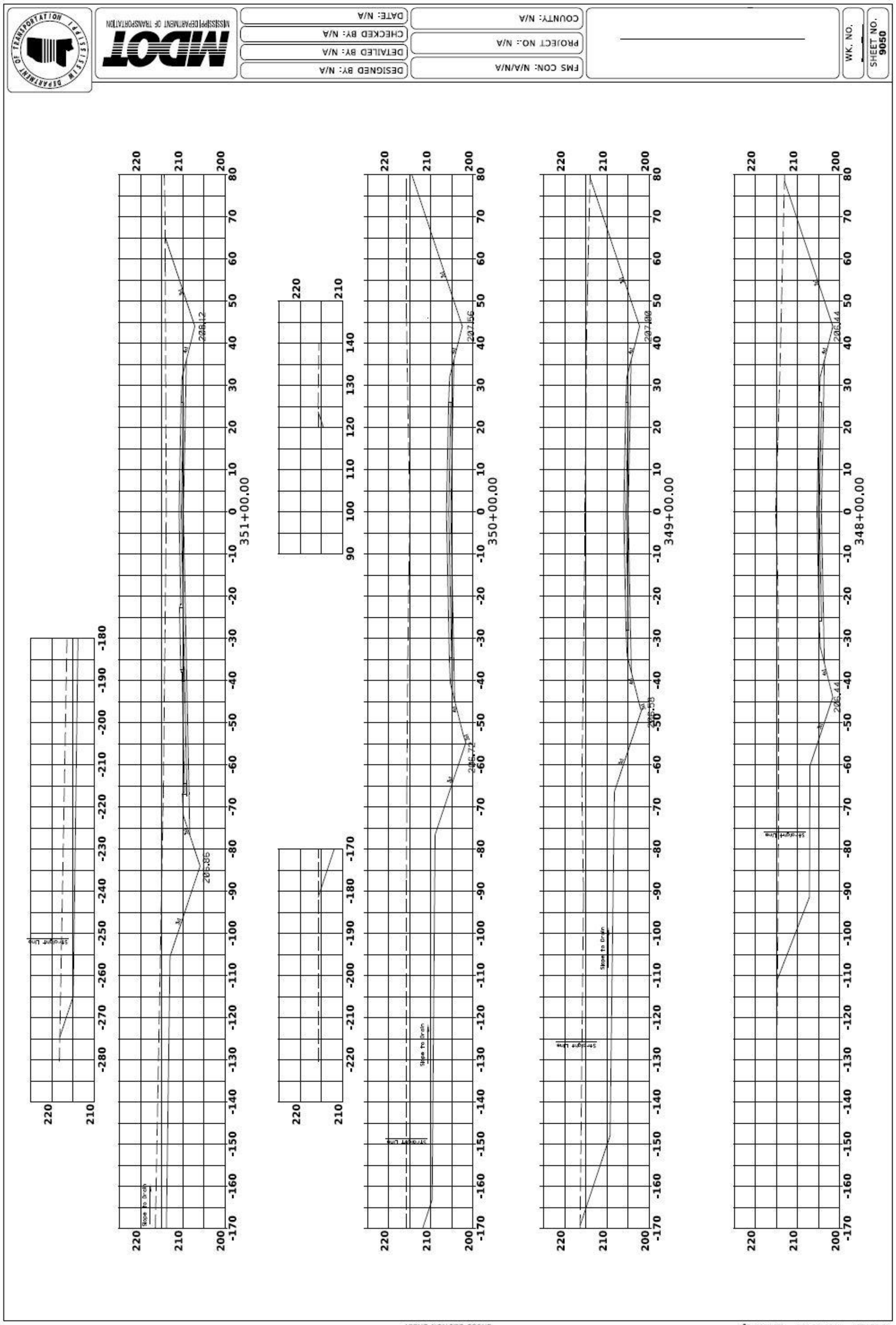
$$X = 0.67'$$

Lengths A to B and A to C should be measured from the design file.

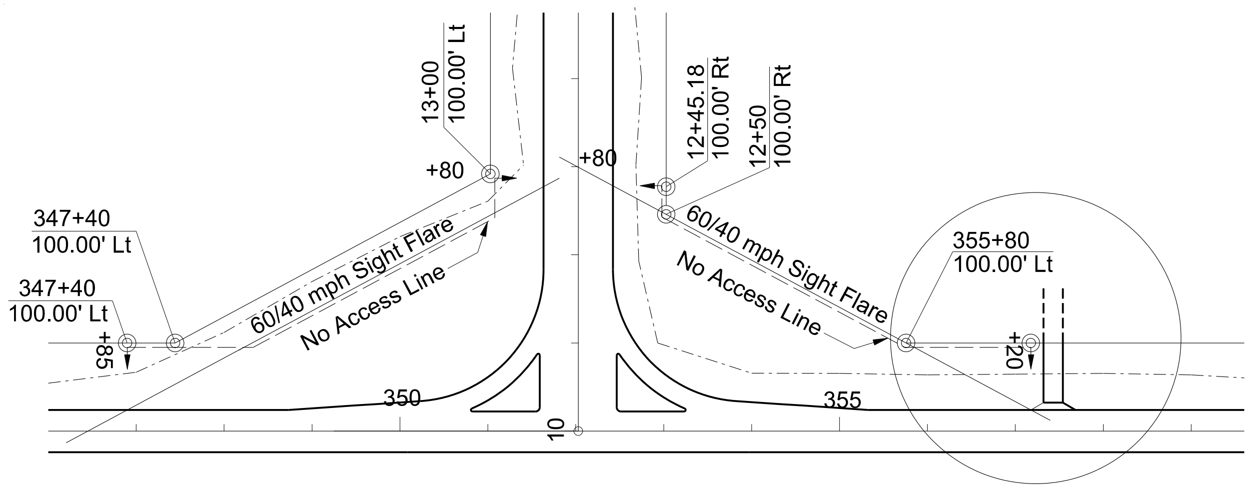
$$Elev_C = 212.99 + X = 213.66$$

Station	Maximum Allowable Elevation
348+00	213.42
349+00	213.66
350+00	213.88
351+00	214.11

EXAMPLE CALCULATION OF SIGHT FLARES
Figure 6-7-E



EXAMPLE CALCULATION OF SIGHT FLARES (Cross Section)
Figure 6-7-F



RECOMMENDED NO ACCESS RIGHT OF WAY
Figure 6-7-G

6-8.0 MEDIAN CROSSOVERS

6-8.01 Warrants/Spacing

The following subsections provide information concerning emergency/maintenance median openings (crossovers) on freeways and crossovers along other divided facilities. The *Access Management Manual* should be reviewed for additional information on the spacing of crossovers.

6-8.01.01 Rural Freeways

The following design criteria apply to all construction/maintenance projects (and to non-project related requests) for rural freeway median crossovers for emergency and law enforcement vehicles and maintenance operations vehicles:

1. Crossovers should only be allowed where interchange spacing exceeds five miles.

2. When interchange spacing exceeds five miles:

   a. Crossovers should be limited in number (spaced no closer than 3-mile intervals when more than one crossover between interchanges is involved).

   b. Crossovers should only be allowed where decision sight distance for an avoidance maneuver for speed/path/direction change on the road is available. Any obstacles (e.g., median cable barrier installations, median guardrail installations) that might impact sight distance should be accounted for during sight distance evaluations.

   c. Crossovers should only be considered in areas where the median width is sufficient to accommodate all types of emergency and maintenance vehicle lengths.

   d. Crossovers should be located no closer than 1500 feet from the end of interchange ramp/loop tapers.

e. Crossovers should be located no closer than 1500 feet from any structure.

f. Crossovers should preferably be located outside of superelevated curves.

g. Crossovers should be constructed in accordance with the latest *Standard Drawings.*

3. Any additional crossovers besides those listed above would be classified as additional maintenance crossovers located at the ends of interchange facilities. The following design criteria apply:

   a. Crossovers should only be allowed for locations where response times related to ice/snow removal (especially on bridges) are significantly impacted due to excessively long travel distances along alternate routes.

   b. Crossovers should comply with Items (2)(b), (2)(c), (2)(d), (2)(e), (2)(f), and (2)(g).

   c. Crossovers should be gated.

6-8.01.02 Urban Freeways

Crossovers for emergency or maintenance purposes are not generally warranted on freeways in urban areas due to the close spacing of interchanges and the extensive development of the abutting roadway network. In a rare case where interchange spacing exceeds five miles in an urban area, the design criteria listed for the rural freeway crossovers apply.

6-8.01.03 Approval Steps for Freeway Crossovers

Requests for new or modified freeway crossovers located off the Interstate Highway System should be submitted to the Chief Engineer for approval. Requests for new or modified freeway crossovers located on the Interstate Highway System should be submitted to the Chief Engineer for initial approval and then to FHWA for final approval.

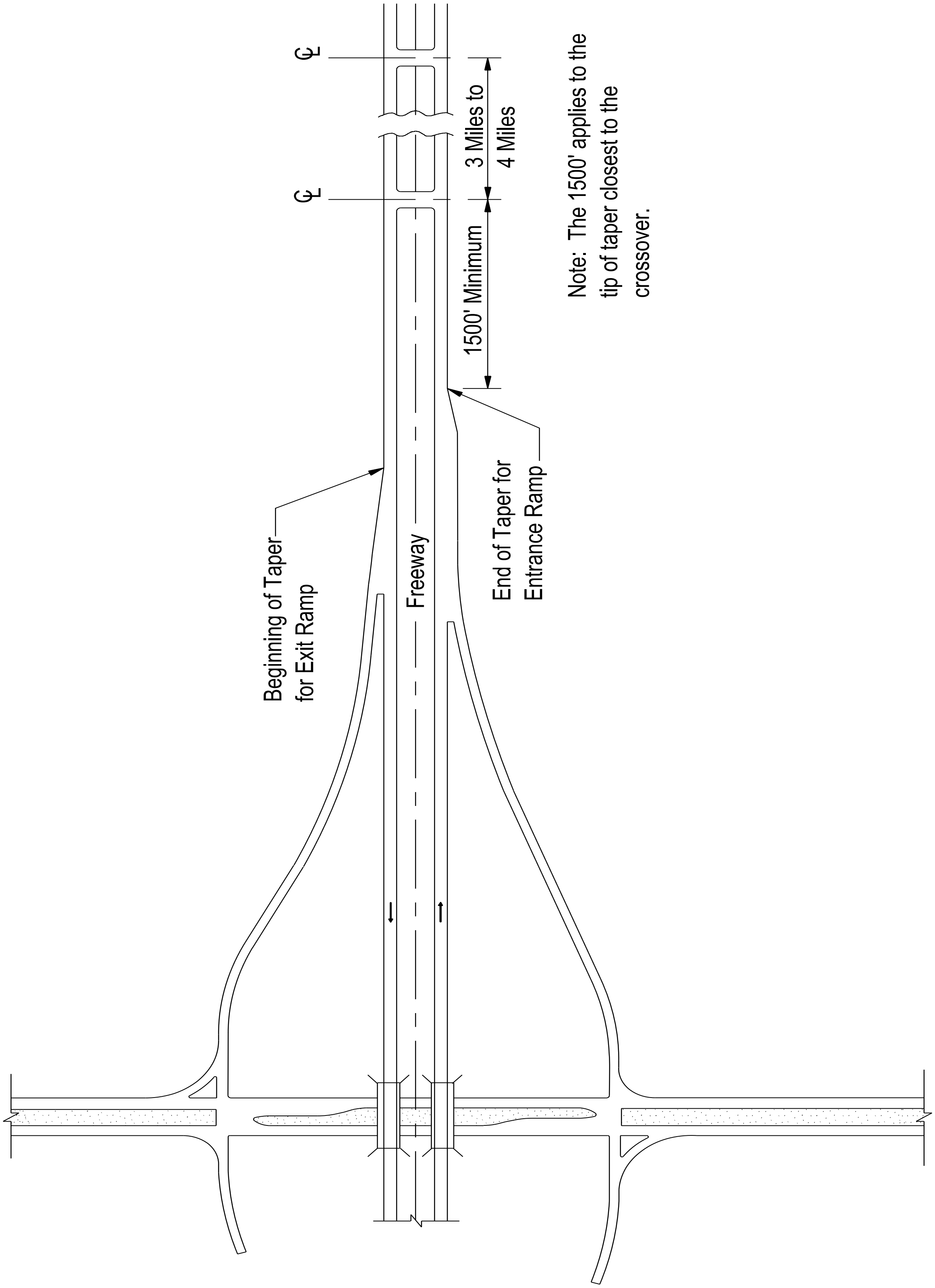
6-8.01.04 Non-Freeways

Crossovers should be provided on all divided roadways with partial control of access (Type 2) or control by regulation (Type 3). According to the *Access Management Manual,* the minimum spacing for full crossovers is 1760 feet, which is measured between the centers of each adjacent crossover in both rural and urban areas, regardless of design speed. The *Access Management Manual* provides additional information on minimum spacing for restricted-access crossovers.

Generally, crossovers on divided roadways are provided for most intersections with existing public roadways, and the spacing for such intersections may often be less than 1760 feet. In such cases, less than the minimum spacing may be allowed where existing roadways with existing traffic patterns of a prominent degree cross the median. However, engineering judgment should be used to determine the proper approach for such intersections that are spaced significantly less than the minimum spacing. Once the crossovers provided for intersections have been determined, those crossovers then become control points for determining spacing and location of intermediate crossovers.

Intermediate crossovers should be spaced as equally as feasible between control points, but may be adjusted to provide reasonable access to properties and businesses and to ensure sufficient sight distance as it relates to the vertical and horizontal alignment. A crossover may be omitted under certain circumstances, such as environmental constraints, limited sight distance, or physical constraints (e.g., bridges).

On new construction projects, the vertical alignment should be checked to ensure it meets minimum SSD at intermediate crossovers, as well as ISD at crossovers for intersecting roadways. However, some projects (e.g., parallel 4-lane projects) may not include reconstruction of the vertical alignment of an existing roadway to meet current SSD criteria. If the vertical alignment of the existing roadway meets 3R criteria, the existing available sight distance may be acceptable. A crossover should not be located at a point that does not meet the minimum SSD values.



LOCATIONS OF MAINTENANCE/EMERGENCY CROSSOVERS ON FREEWAYS
Figure 6-8-A

6-8.02 Design Criteria for Median Crossovers

Figures 6-8-B, 6-8-C, 6-8-D, and 6-8-E provide the design criteria for median crossovers for various median widths. Section 6-3.0 should be referenced for deceleration length and storage length at crossovers with intersections.

6-8.02.01 Turning Radii

Crossovers should be designed to accommodate left-turning vehicles. Turning radii for the intersection layout should be based on the selection of the design vehicle, turning characteristics of the design vehicle, acceptable encroachment, and angle of turn. The discussion in Section 6-2.0 on these factors also applies to crossovers.

The criteria in Figures 6-8-C, 6-8-D, and 6-8-E will accommodate most design vehicles. For narrow medians, the criteria in Figure 6-8-B should be used to determine the length of crossovers (L) for the selected design vehicle. These criteria should allow the vehicle to make the left turn without encroachment onto the adjacent travel lane.

6-8.02.02 Shape of Median Ends

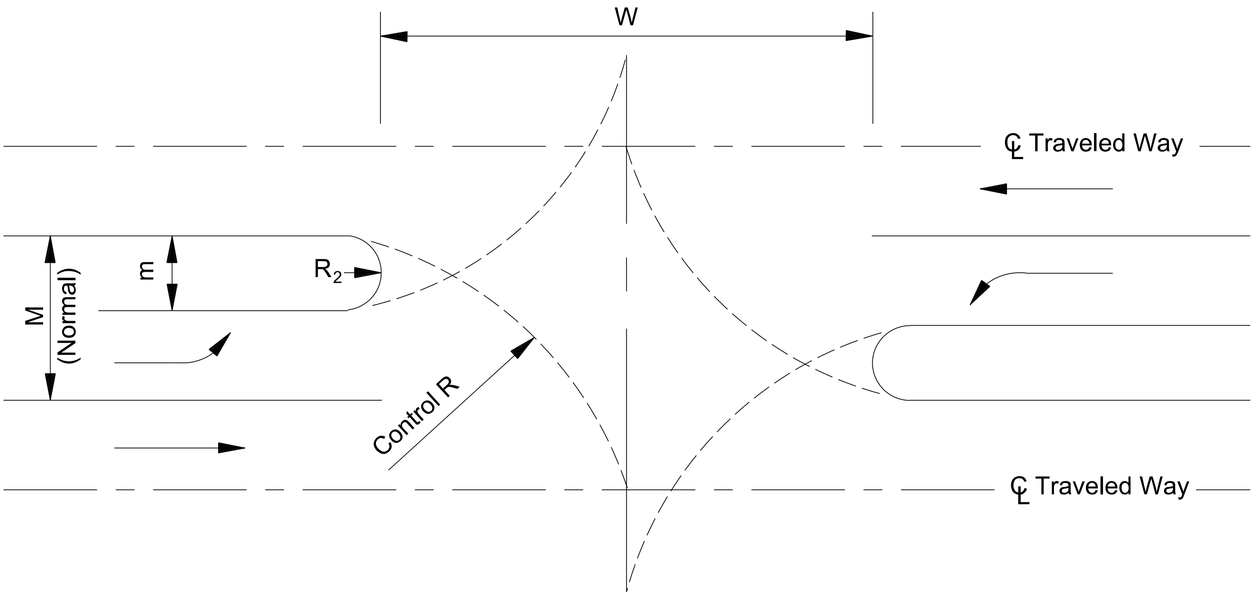
The shape of median ends at crossovers depends on the width of the divider (m) between the left-turn and the opposing inside travel lane, or between the two inside travel lanes for openings where there is no left-turn lane. This width is in contrast to the median width (M), which is measured between the edges of the two inside travel lanes and, therefore, includes the width of left-turn lanes, if present.

The most common types of median ends are the semicircular end and the bullet-nose end. Recommended criteria for the selection of the median end shape based on "m" are provided in Table 6-8-A.

Figure 6-8-B provides recommended design criteria for the semicircular end, and Figure 6-8-C provides design criteria for the bullet-nose end. Figure 6-8-D provides typical crossovers for 40-foot and 64-foot medians. Crossovers for median widths greater than 64 feet should be treated as separate intersections. Figure 6-8-E provides typical crossovers for median widths greater than 64 feet.

Table 6-8-A
SELECTION OF MEDIAN END SHAPE

m	Median End Shape
$m < 10$ ft	Semicircular
10 ft $\leq m \leq$ 64 ft	Bullet Nose
$m > 64$ ft	Treated as separate intersections



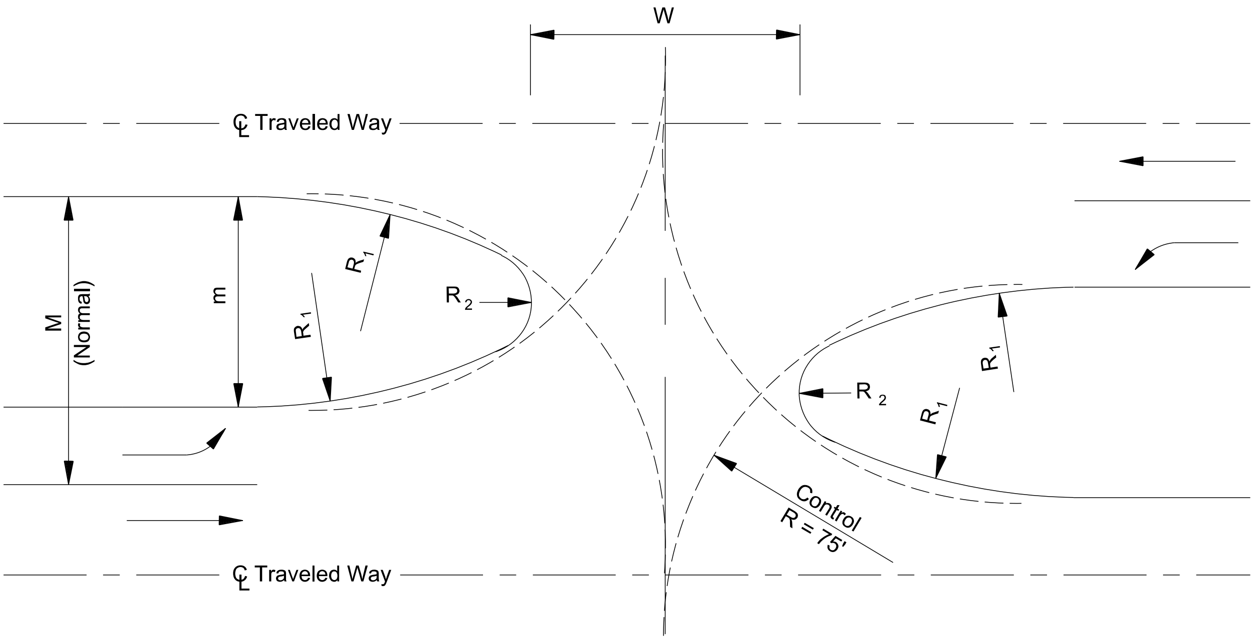
Key: *M* = *Median width measured between the two edges of the inside travel lanes.*
m = *Width of divider (raised or flush) remaining after the widths of the left-turn lane (if any) and/or shoulders have been subtracted from the median width, M.*
W = *Width of crossover to accommodate the turning path of the design vehicle.*

m (ft)	W (ft)	
	S-BUS-40	**WB-40**
4	96	146
6	94	144
8	92	142
10	90	140

Notes:

1. *According to Table 6-8-A, the semicircular end should be used when m is less than 10 feet.*
2. *Control R is dependent upon the turning radii of the design vehicle selected.*
3. *R2 = m/2*
4. *Typically, the S-BUS-40 design vehicle should be used for these crossovers.*
5. *These suggested criteria should only be used as starting points for the design of the crossover. See the discussion in Section 6-8.02.03 for more information on the design of the crossover.*
6. *Sign placement should be considered to ensure 2-foot horizontal clearance from edge of sign.*

MEDIAN CROSSOVERS
(Semicircular End)
Figure 6-8-B



Key:

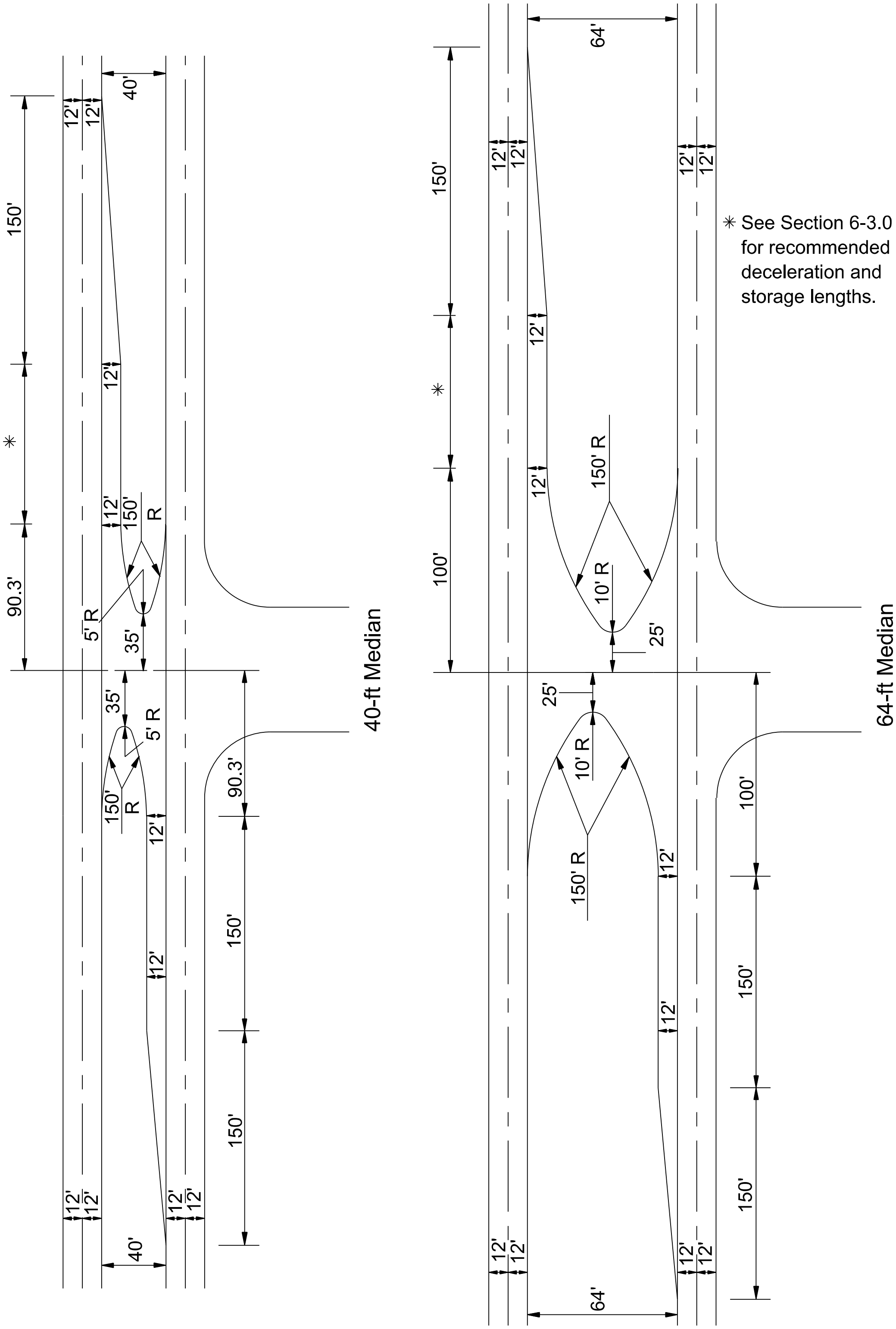
- *M* = *Median width measured between the two edges of the inside travel lanes.*
- *m* = *Width of divider (raised or flush) remaining after the widths of the left-turn lane (if any) and/or shoulders have been subtracted from the median width, M.*
- *W* = *Width of crossover to accommodate the turning path of the design vehicle.*

m (ft)	L (ft)
10	104
12	98
14	92
16	88
20	78
24	72
28	65
32	60
36	54
40	49
≥ 54	44

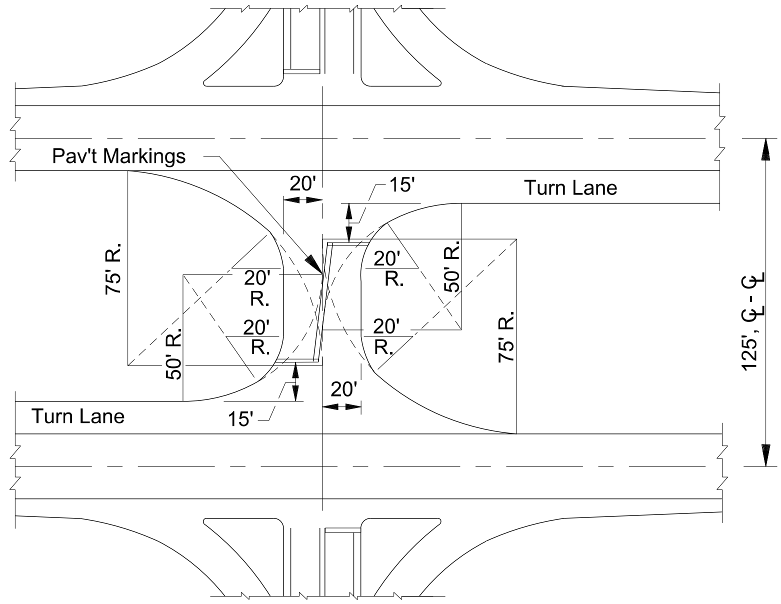
Notes:

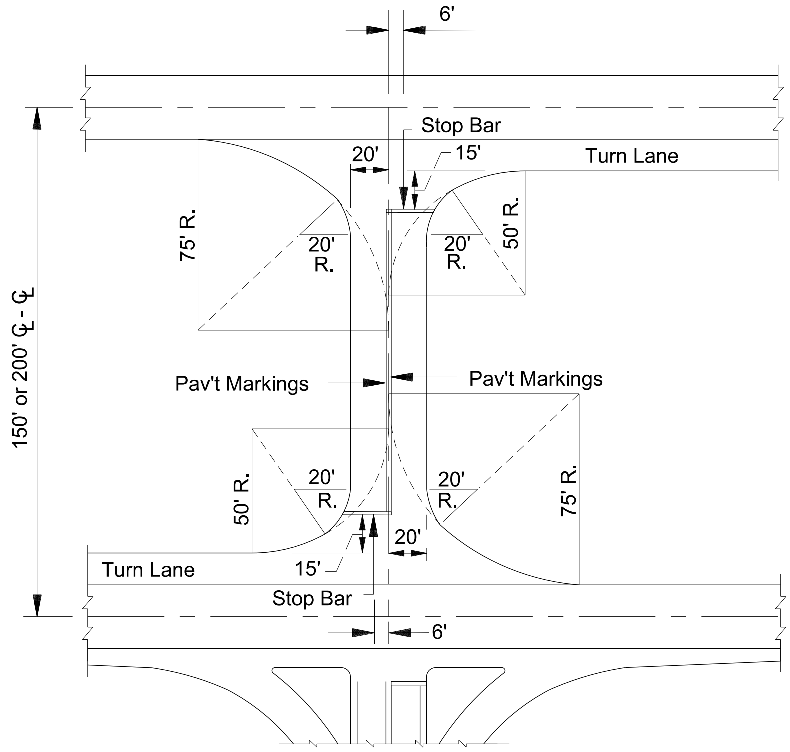
1. *According to Table 6-8-A, the bullet nose should be typically used when 10 feet $\leq m \leq 64$ feet.*
2. *$R_2 \cong m/5$*
3. *These suggested criteria should only be used as starting points for the design of the median crossover, which are based on a S-BUS-40 design vehicle. See the discussion in Section 6-8.02.03 for more information.*
4. *R_1 = 150 feet*

MEDIAN CROSSOVERS
(Bullet-Nose End)
Figure 6-8-C

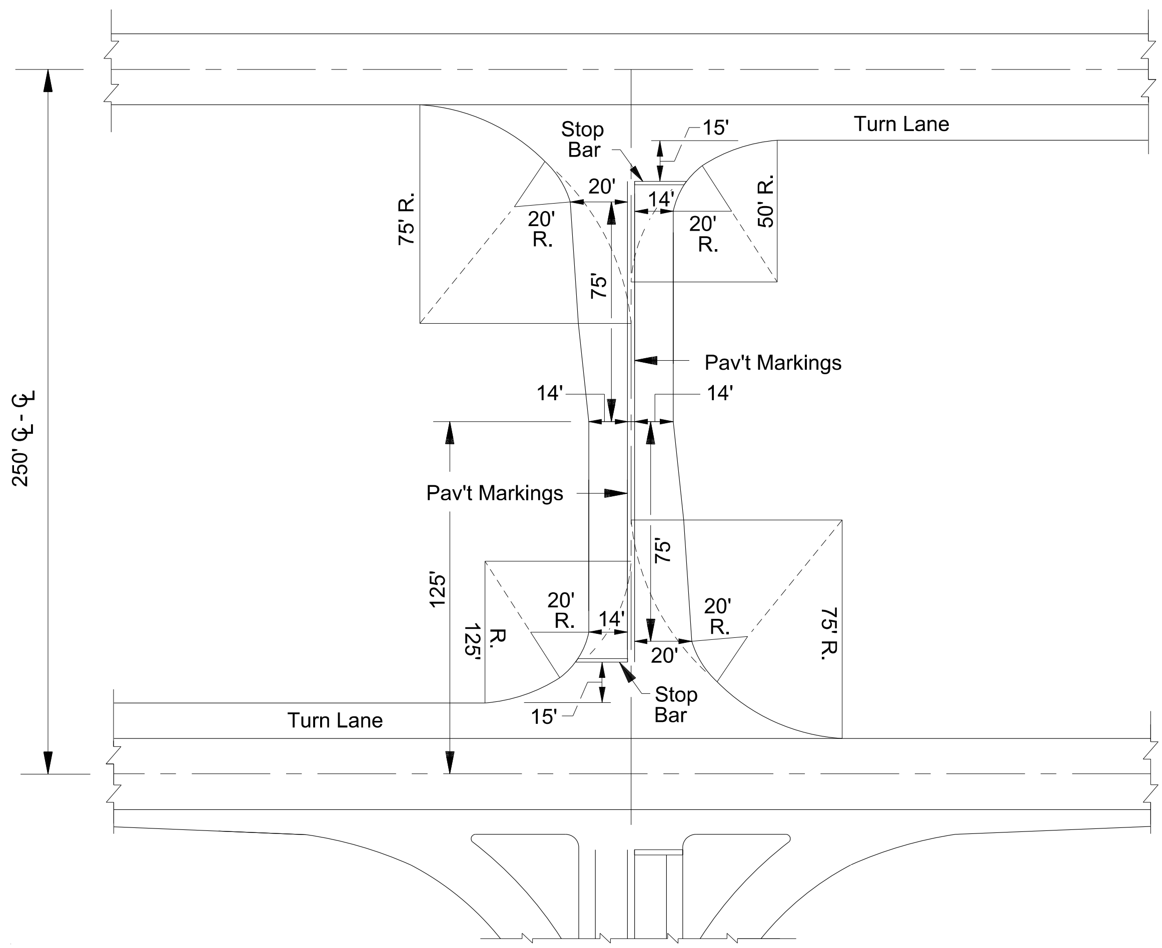


TYPICAL CROSSOVERS AT INTERSECTIONS (M ≤ 64 feet)
Figure 6-8-D





TYPICAL CROSSOVERS WITH OR WITHOUT INTERSECTIONS
Figure 6-8-E



TYPICAL CROSSOVERS WITH OR WITHOUT INTERSECTIONS
(Continued)
Figure 6-8-E

6-8.02.03 Width of Crossover

The width of a crossover should properly accommodate the turning path of the design vehicle. The width of a crossover for any divided roadway should be greater than or equal to the width of the intersecting road. Recommended widths of crossovers are presented in Figure 6-8-B for the semicircular end and Figure 6-8-C for the bullet-nose end. The table values should only be used as a starting point. Each crossover should be evaluated individually to determine its proper design. The following factors should be considered in the evaluation:

1. Turning Radius – The design should be checked to ensure the turning radius design can adequately accommodate the selected design vehicle. Consideration should be given to the frequency of the turn and to the encroachment onto adjacent travel lanes or shoulders by the turning vehicle.

2. Nose Offset – At 4-legged intersections, traffic going straight through the crossover will pass the nose of the median end (semicircular or bullet nose). The offset between the nose end and through travel lane (extended) should be a minimum of two feet.

3. Intersection Symmetry – The three or four legs of the intersection may have various combinations of through travel lanes and turn lanes. The overall symmetry of the intersection should be considered, which in turn, may affect the design of the crossover.

4. Location of Crosswalks – Pedestrian crosswalks should intersect the median to provide some refuge for pedestrians. Therefore, the crossover design should be coordinated with the location of crosswalks. Where medians are intended to also provide pedestrian refuge, they should be sized and positioned to provide a clear pedestrian path that has a minimum width of six feet and a minimum length of six feet. The path shall be clear of all obstructions (e.g., curbs, poles, sign posts, utility boxes).

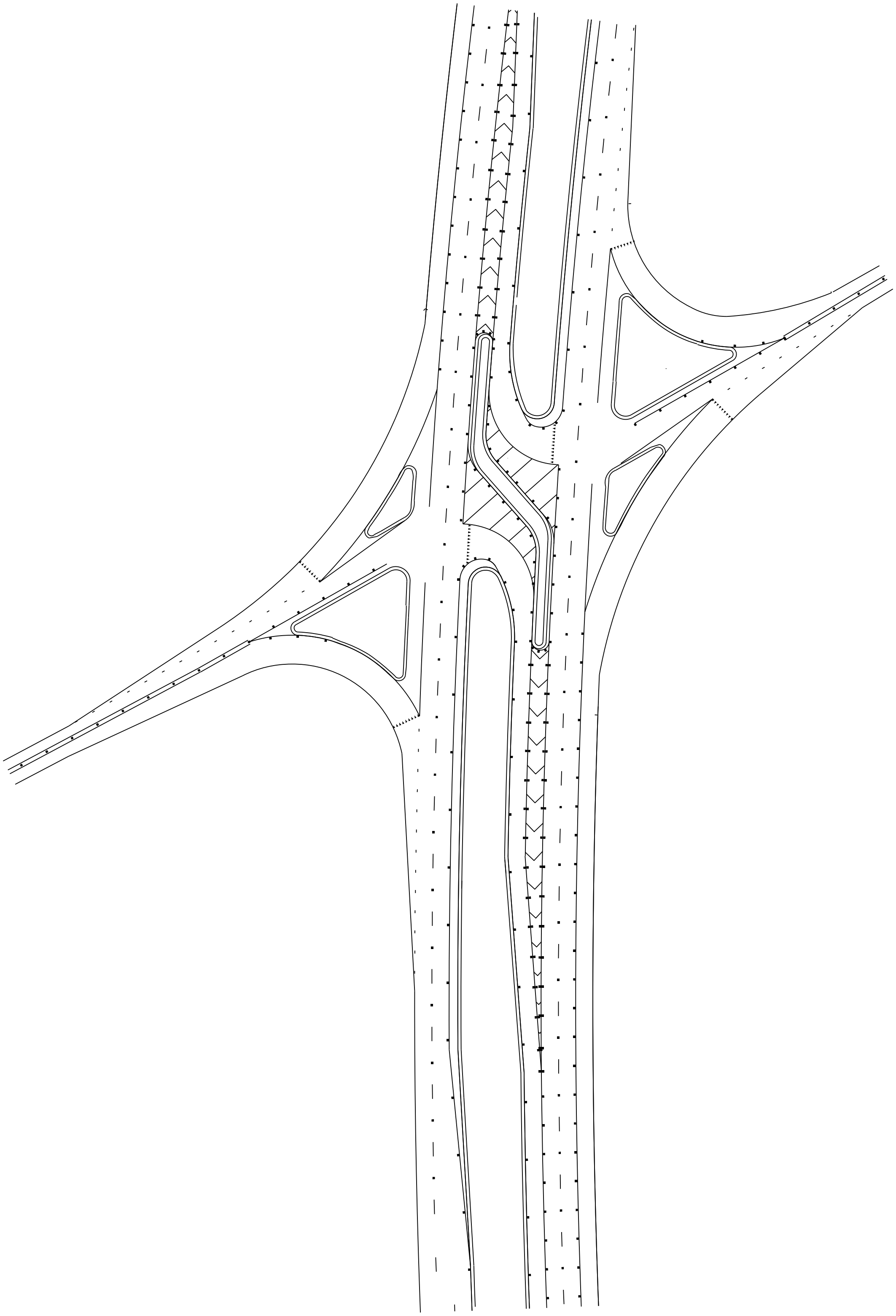
6-8.02.04 U-turns

Crossovers designed to accommodate U-turns are not typically provided for narrow medians. Preferably, a vehicle should be able to begin and end the U-turn on the inner lanes next to the median, but designing for the vehicle to end the U-turn on the outer lanes is acceptable. This design also allows a stopped vehicle to be fully protected within the median area. Figure 6-8-G provides the minimum recommended median widths for U-turn maneuvers and various design vehicles.

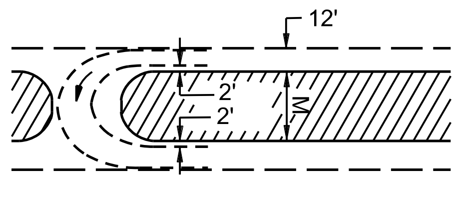
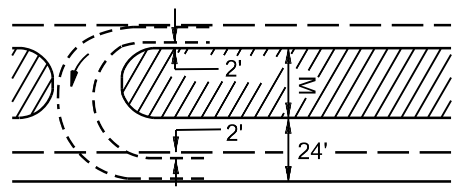
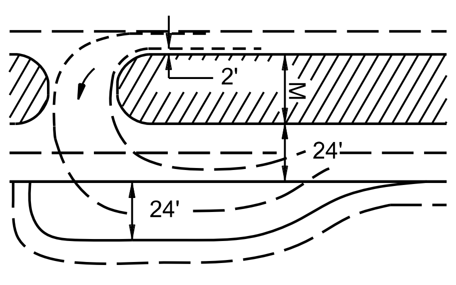
Where U-turns are to be provided within a narrow median, a bulb-out as shown in Figure 6-8-G should preferably be designed to provide adequate room for large vehicles to make the turn. The bulb-out pavement should be designed to accommodate the selected design vehicle. To deter vehicles from stopping on the through lanes, a left-turn lane with proper storage capacity should be provided to accommodate turning vehicles.

6-8.02.05 Skewed Intersections

Median crossovers should desirably not be skewed. Skewed median crossovers typically have larger openings, which may result in special channelization and other adjustments to the left-turn lanes. Each skewed design should be designed separately to determine the appropriate layout.



TYPICAL CROSSOVERS WITH J-TURN DESIGN
Figure 6-8-F

Type of Maneuver		M – Minimum Width of Median (ft) for Design Vehicle				
		P	SU-40	BUS	WB-62	WB-67
		Length of Design Vehicle (ft)				
		19	40	40	63	68
Inner Lane to Inner Lane		30	76	63	69	69
Inner Lane to Outer Lane		18	64	51	57	57
Inner Lane to Shoulder		8	54	41	47	47

MINIMUM DESIGN FOR U-TURNS
Figure 6-8-G

6-9.0 RURAL DRIVEWAYS

Table 6-9-A and Figure 6-9-A present the design criteria for rural driveways. Section 14-2.07 presents criteria for urban driveways. The *Standard Drawings* and *Access Management Manual* present other design details that should be considered. In addition, the following Department rules are related to driveways:

- Rule No. 941-7501-03001 “Processing Permit Applications”
- Rule No. 941-7501-03002 “Construction and Maintenance of Driveway, County Road and Municipal Street Connections to State Highways”
- Rule No. 941-7501-04013 “Driveway and Street Connections, Median Openings, Frontage Roads”

Table 6-9-A
RECOMMENDED DESIGN CRITERIA FOR RURAL DRIVEWAYS

Driveway Design Element		Driveway Type	Design Criteria
Minimum Corner Spacing (C)		All	125 ft **(1)**
Turning Radii (R)		Residential	10 ft
		Commercial/ Industrial	30 ft – 50 ft
Width (W) **(2)**		Residential	Minimum: 16 ft
		Commercial/ Industrial	Minimum: 30 ft Maximum: 50 ft
Maximum Grades	G_1	All	Cut Section: 2%-4% Fill Section: Shoulder Slope (4% Typical) (down)
	G_2	All	10%
Change in Grade without Vertical Curve (ΔG)		All	Maximum: 12%
Driveway Side Slopes **(3)**	Within Clear Zone	All	$V \geq 50$ mph — Des: 10:1 Max: 6:1 $V \leq 45$ mph — Max: 3:1
	Outside Clear Zone	All	Maximum: 3:1
Sight Distance		All	**(4)**

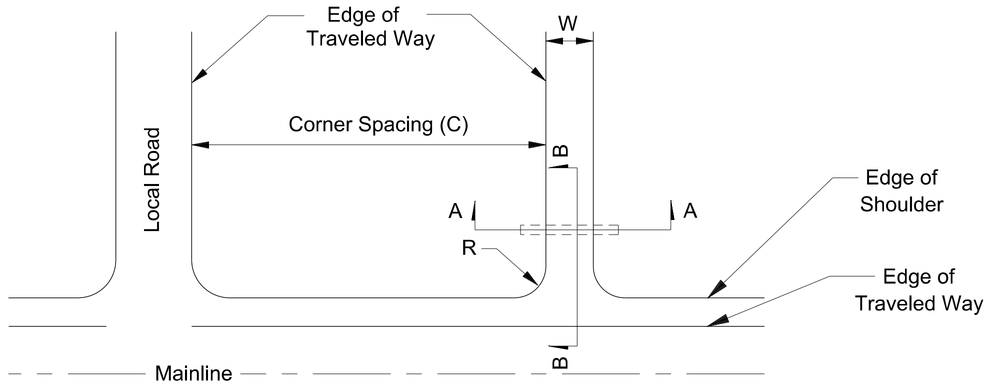
Note: Illustrations of terms are shown in Figure 6-9-A.

1. <u>*Corner Spacing (C)*</u> *– No part of a driveway entrance or exit is permitted within the turning radius of an intersection (see Section 6-2.0) or within the sight triangles (flares) of an intersection (see Section 6-7.04). See* Access Management Manual *for more information on corner clearance.*

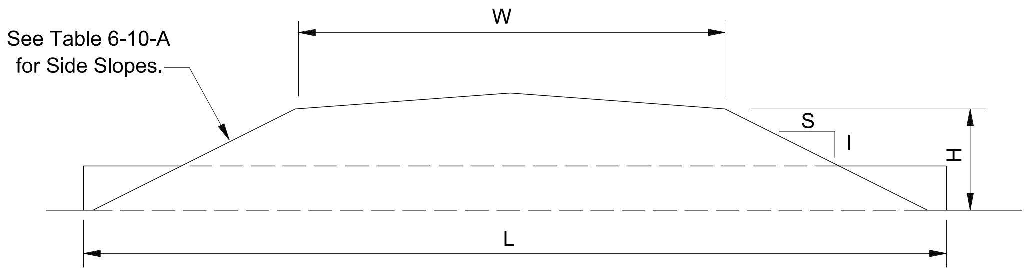
2. <u>*Widths*</u> *– For 2-way operation, the minimum width for multi-unit residential driveways should be 30 feet. For commercial/industrial driveways with 2-way operation, widths should be provided based on the likelihood that vehicles will be entering and exiting from the driveway simultaneously.*

3. <u>*Driveway Side Slopes*</u> *– Section 9-3.06 discusses the transverse sideslopes of driveways located within the clear zone of the roadway.*

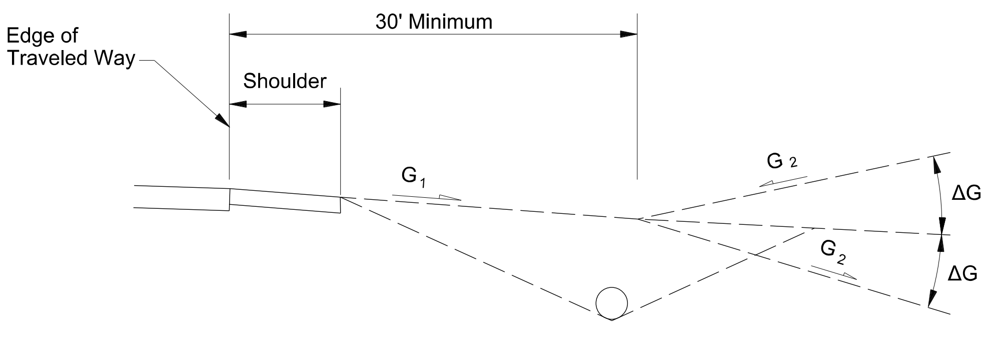
4. <u>*Sight Distance*</u> *– Driveways should be designed so that the approach to the driveway meets or exceeds the minimum SSD. Sight obstructions in the vicinity of the driveway entrance (e.g., large trees, hedgerows) that may inhibit sight distance should be checked. To perform the check, it is reasonable to assume an eye location of approximately 10 feet from the edge of traveled way.*



Plan View



Section A-A



Section B-B

(For Cut And Fill Section)

Example 6-9-1

Given:	W	=	16 feet
	H	=	9 feet
	S	=	6

Problem: Determine length of pipe.

Solution: L = 16 feet + 2(6)(9 feet) = 124 feet
Round to next increment of 8 feet, which equals 128 feet.

TYPICAL RURAL DRIVEWAYS
Figure 6-9-A

6-10.0 RAILROAD CROSSINGS

The following subsections present geometric design criteria for at-grade railroad crossings. For other design considerations (e.g., traffic control devices, railroad crossing surfaces), coordination with the Highway and Rail Safety Division is necessary. In addition, the *FHWA Railroad-Highway Grade Crossing Handbook* should be reviewed for more information. Railroad diagnostic surveys may also be required.

6-10.01 <u>Responsibilities</u>

The railroad company is responsible for all work necessary for the adjustment of its tracks to meet altered or established roadway grades, and the railroad should construct the roadway grade crossing as indicated on the plans. The railroad company should also install flashing light signals, automatic gates, guardrails, and other warning devices as recommended by the Department. All work performed by the railroad company should be conducted under an agreement between the railroad company and the Department. The agreement should be on a force account basis or with a contract that allows the Department to reimburse the railroad company for allowable construction costs.

Two sets of Field Inspection Plans should be submitted to the Highway and Rail Safety Division's Rails Engineer as soon as they are available. A railroad representative should participate in the Field Inspection. The Rails Engineer should be responsible for negotiations with the railroad company and should prepare the railroad agreement, as applicable. Following the Office Review, the Roadway Design Division should prepare draft final plans and submit plan copies to the Rails Engineer for further processing with the railroad company.

If a project includes a railroad right of way easement or Use Permit, the Right of Way Division should prepare the necessary documents and coordinate this effort with the railroad company.

6-10.02 <u>Design</u>

6-10.02.01 Sight Distance

Two sight distance applications should be addressed at railroad/roadway grade crossings:

- <u>Case A</u> – Sight distance to allow a driver to either pass through the grade crossing prior to the train's arrival or to stop the vehicle prior to encroachment in the crossing area.

- <u>Case B</u> – Sight distance for a driver that is stopped at the crossing and then decides to cross the tracks.

Figures 6-10-A and 6-10-B illustrate the conditions under which Case A and Case B sight distances should apply. Table 6-10-A presents the sight distances that are recommended for both Case A and Case B. If actual field conditions differ from those presented in Figures 6-10-A and 6-10-B (e.g., skewed crossings, more than one set of tracks), AASHTO's *A Policy on Geometric Design of Highways and Streets* should be referenced to adjust the criteria in Table 6-10-A.

The sight distances for Case B should be available at all railroad/roadway crossings that are not equipped with automatic gates. Case A is important at crossings with only passive control devices (i.e., crossbucks, pavement markings, advance warning signs). Where feasible, the criteria for Case A should also be met at crossings with active control devices (i.e., flashing lights, automatic gates). However, the Case A sight distances may be difficult to attain in the field. Where these distances are not feasible, the following potential countermeasures should be considered:

- installing active warning devices where only passive devices currently exist
- employing speed control signs, flashing advance warning lights, and other devices to lower approaching vehicular speeds to be consistent with the available sight distance
- forcing all vehicles to a complete stop

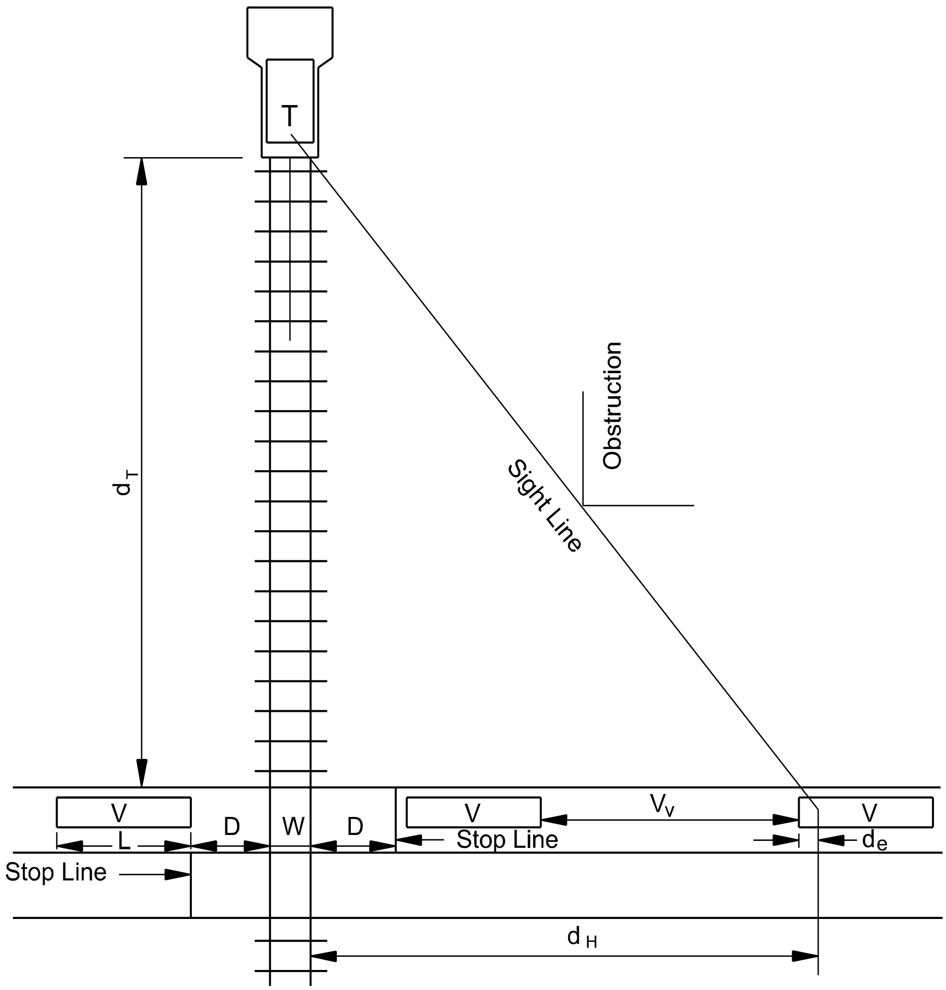
6-10.02.02 Horizontal Alignment

Where feasible, the alignment of the roadway and railroad crossing should intersect at an angle of 90 degrees, and neither the roadway nor the railroad should be in a horizontal curve. Meeting these objectives should enhance driver safety and comfort, and also reduce maintenance. Chapter 3, “Horizontal Alignment”, should be reviewed to determine the roadway horizontal alignment criteria.

6-10.02.03 Vertical Alignment

The roadway should be relatively level where it crosses the railroad. Where vertical curves are provided, they should meet the vertical alignment criteria presented in Chapter 4, “Vertical Alignment.”

Figure 6-10-C presents the minimum design for vertical alignment at railroad crossings to prevent low-clearance vehicles from bottoming out on the tracks. Adjustments may be recommended where railroad track superelevation exists at the crossing.



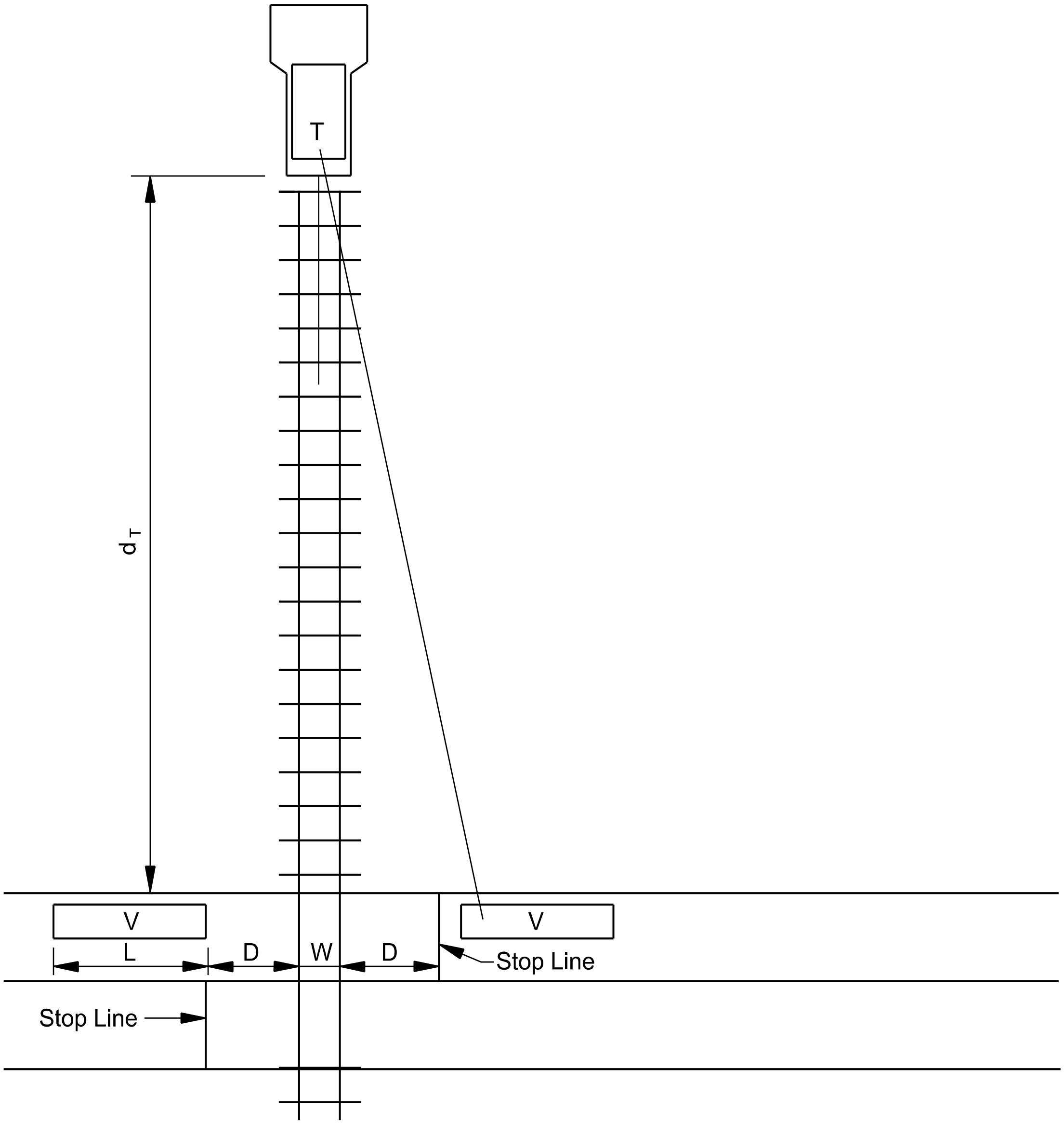
d_T = Sight distance leg along the railroad tracks to permit the maneuvers described for d_H (feet)

V_T = Velocity of train (miles per hour)

d_H = Sight distance leg along the roadway allowing a vehicle proceeding to speed V_V to cross tracks safely even though a train is observed at a distance d_T from the crossing or to safely stop the vehicle without encroachment of the crossing area (feet)

V_V = Velocity of vehicle (miles per hour)

CASE A: APPROACHING VEHICLE TO SAFELY CROSS OR STOP AT RAILROAD CROSSING
Figure 6-10-A



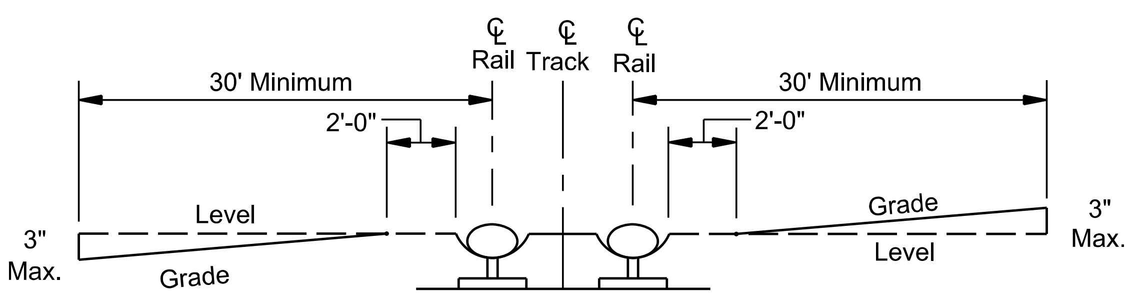
d_T = Sight distance along railroad tracks (feet)

V_T = Velocity of train (miles per hour)

CASE B: DEPARTURE OF VEHICLE FROM STOPPED POSITION TO CROSS SINGLE RAILROAD TRACK
Figure 6-10-B

Table 6-10-A
SIGHT DISTANCE AT RAILROAD CROSSINGS

Train Speed (mph)	Case B Departure from stop	Case A Moving Vehicle					
	Vehicle Speed (mph)						
	0	10	20	30	40	50	60
	Distance Along Railroad From Crossing, d_T (ft)						
10	255	155	110	102	102	106	112
20	509	310	220	203	205	213	225
30	794	465	331	305	307	319	337
40	1019	619	441	407	409	426	450
50	1273	774	551	509	511	532	562
60	1528	929	661	610	614	639	675
70	1783	1084	771	712	716	745	787
80	2037	1239	882	814	818	852	899
90	2292	1394	992	915	920	958	1012
	Distance Along Roadway from Crossing, d_H (ft)						
All		69	135	220	324	447	589



PROFILE AT RAILROAD/ROADWAY GRADE CROSSINGS
Figure 6-10-C

6-10.02.04 Intersection Storage across Tracks

The following information applies to all roadway improvement projects where the route is adjacent and parallel to a railroad. Where an at-grade railroad crossing is near an intersection (signal- or stop-controlled), the crossing should be designed to keep vehicles from stopping or storing on the tracks. The following factors should be identified and considered during the planning stage:

1. Clear Storage Distance – The distance available for vehicle storage should be between a point six feet from the rail nearest the intersection to the intersection stop line or the normal stopping point on the roadway. At skewed crossings and intersections, this 6-foot distance should be measured perpendicular to the nearest rail either along the centerline or edge line of the roadway, as appropriate, to obtain the shorter clear distance. Where exit gates are used, the distance available for vehicle storage should be measured from a point clear of the exit gate. Where the exit gate arm is not perpendicular to the roadway, clearance should be either along the centerline or edge line of the roadway, as appropriate, to obtain the shorter clear distance.

2. Space for Vehicular Escape – Providing an escape area for vehicles (e.g., shoulder, an area beyond the curb and gutter, flush medians, flush-corner islands, right-turn acceleration lanes, improved corner radii) should be considered on the far side of any crossing.

3. Traffic Signals Operations Near Railroad/Roadway Crossings – Where traffic control signals are proposed to control road users at roadway-rail grade crossings or at intersections near railroad crossings, coordination with the Highway and Rail Safety Division is necessary.

4. Restricted Intersection Capacity – If there are periods of frequent railroad preemption of traffic signals, the effects of reduced traffic flow, lack of progression on the roadway paralleling the tracks, and traffic backups should be analyzed. Available computer programs should be used to analyze different capacity and operational scenarios and to recommend any countermeasures.

5. Conflicting Commercial Access – Left-turn vehicular movements into businesses that may inhibit the clearance of queued traffic on the approaches to railroad tracks should be discouraged. Design features that would eliminate the problems (e.g., left-turn lane, raised median) should be considered.

6. Protected Left-Turn Storage – On roadways that parallel tracks, the recommended storage for left turns into the side roadway and across the tracks during pre-emption of the traffic signals should be analyzed. Backups into the through lanes may result if the proper storage length is not available.

7. Side Street Left-Turn Lane Capacity – On roadways that cross railroad tracks, sufficient left-turn storage lengths should be provided that avoid left turns queuing onto and blocking the through lanes.

8. Sight Restrictions – Sight distance triangles along railroad tracks should be analyzed, and any obstructions should be eliminated. Information on this analysis can be found in Section 6-6.0.

6-10.02.05 Intersection Storage for Auxiliary Lanes Parallel to the Tracks

On roadways that run parallel to the railroad and where an actuated "NO RIGHT TURN" sign is proposed in conjunction with railroad preemption, a right-turn lane for the right-turn movement across the tracks should be considered. Auxiliary lanes provide refuge for right-turning vehicles during railroad preemption and eliminate traffic temporarily blocking the through lanes.

6-11.0 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets,* AASHTO, 2018.

2. *Highway Capacity Manual 2010*, Transportation Research Board, 2010.

3. *Manual on Uniform Traffic Control Devices*, National Advisory Committee on Uniform Traffic Control Devices, 2009.

4. NCHRP Report 457, *Evaluating Intersection Improvements,* 2001

5. NCHRP Report 672, *Roundabouts: An Informational Guide – Second Edition*, Transportation Research Board, 2010.

6. Highway Research Board, Special Report No. 74, *Channelization*, National Research Council, 1962.

7. Report No. FHWA-RD-76-86, *Technical Guidelines for the Control of Direct-Access to Arterial Highways*, August 1975.

8. NCHRP 93, *Guidelines for Medial and Marginal Access Control on Major Roadways*, Highway Research Board, 1970.

9. *Highway Safety Design and Operations Guide*, AASHTO, 1997.

10. "Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections," M.D. Harmelink, Highway Research Record 211, 1967.

11. NCHRP Report 279, *Intersection Channelization Design Guide*, Transportation Research Board, November 1985.

12. NCHRP Report 375, *Median Intersection Design*, Transportation Research Board, 1995.

13. NCHRP Synthesis 225, *Left-Turn Treatments at Intersections*, Transportation Research Board, 1996.

14. NCHRP Report 383, *Intersection Sight Distance*, Transportation Research Board, 1996.

15. Technical Council Informational Report, "Design and Use of Two-Way Left-Turn Lanes," Institute of Transportation Engineers, ITE Journal, February 1981.

16. Technical Council Informational Report, “Design Criteria for Left-Turn Channelization,” Institute of Transportation Engineers, ITE Journal, February 1981.

17. Technical Council Informational Report, *Effectiveness of Median Storage and Acceleration Lanes for Left-Turning Vehicles*, Institute of Transportation Engineers, ITE Journal, March 1985.

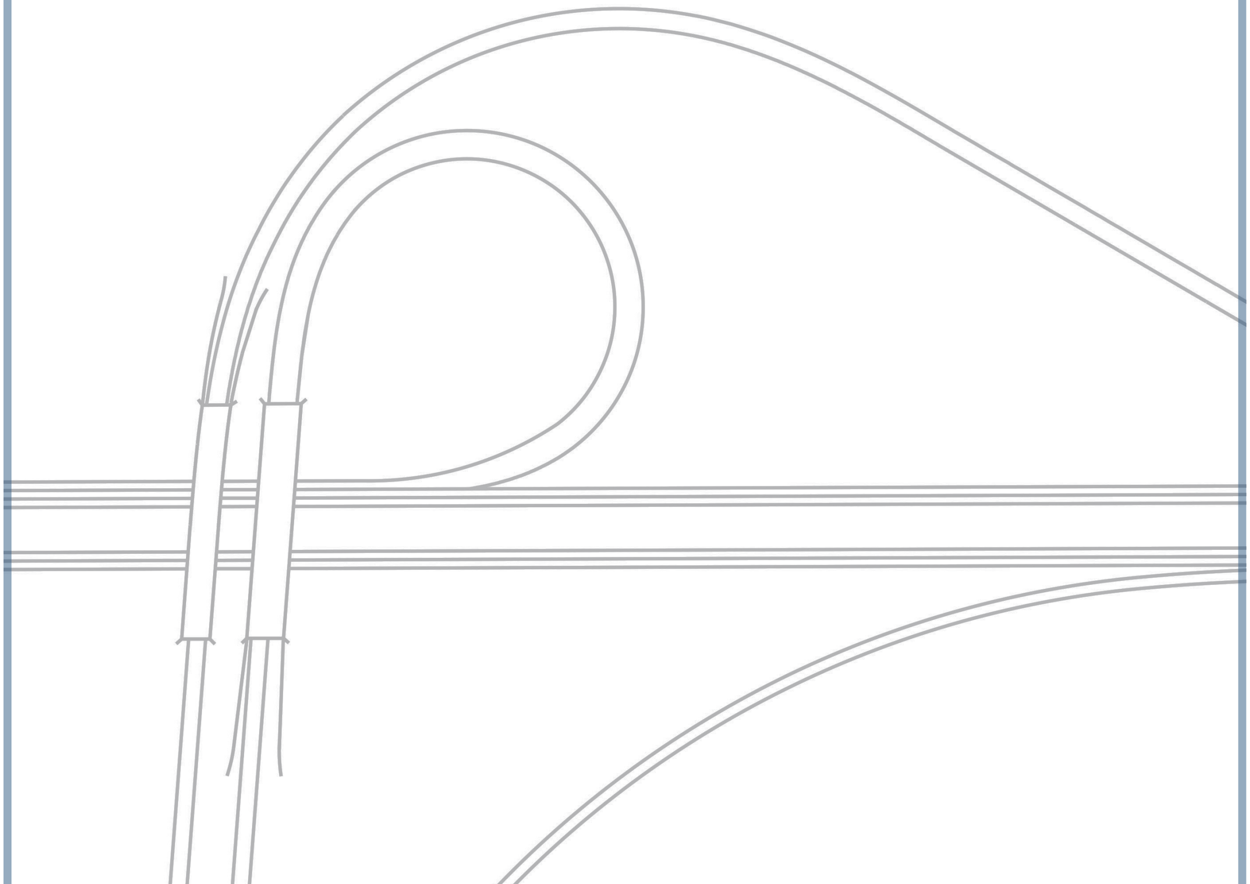
18. “Development of Guidelines for the Application of Continuous Two-Way Left-Turn Median Lanes,” Final Report — EES470, Zolton Nemath, Ohio State University, July 1976.

19. *Access Management Manual*, Mississippi Department of Transportation, 2010.

20. Institute of Transportation Engineers Recommended Practice, *Guidelines for Driveway Design and Location*, Institute of Transportation Engineers, 1987.

21. *Driveway and Street Intersection Spacing*, Transportation Research Circular 456, Transportation Research Board, March 1996.

22. *Preemption of Traffic Signals near Railroad Crossings: An ITE Recommended Practice*, Institute of Transportation Engineers, 2006.



CHAPTER 7

Grade Separations and Interchanges

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Chapter 7
GRADE SEPARATIONS AND INTERCHANGES

Grade separations are provided to allow two transportation facilities to cross at different levels (e.g., roadways, railroads, pedestrian crossings). Separations are defined in terms of the major roadway (mainline) crossing over (overpass) or under (underpass) the minor roadway (crossroad). The type of bridge structure provided at grade separations should be based upon site conditions and span lengths required to obtain the recommended horizontal and vertical clearances.

An interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provides for movement of traffic between two or more roadways on different levels. The operational efficiency, capacity, safety, and cost of the roadway facility are largely dependent upon its design.

On roadways with partial control of access, intersections with public roads are usually accommodated by an interchange or an at-grade intersection. Grade separations that do not allow access are not normally provided. Typically, an interchange will be selected for higher-volume intersecting roads. Therefore, on a roadway with partial control of access, the decision to provide an interchange will be, in general, based on further warrants.

Once a decision has been made to fully control the access of a roadway, each intersecting roadway must be terminated, rerouted, provided a grade separation, or provided an interchange. The continuity of the crossroad, feasibility of an alternative route, and anticipated demand for access to the crossroad should be considered when evaluating a proposed grade separation or interchange.

This chapter provides information on the design of grade separations and interchanges, including interchange types and selection, ramps, operations, and spacing.

7-1.0 GRADE SEPARATIONS

7-1.01 <u>General Design Considerations</u>

The decision on whether to provide an overpass or an underpass is normally dictated by topography, roadway classification, and cost. If the topography does not favor one alternative over the other, the alternative that provides a preferable design for the mainline should typically be selected.

Consideration should also be given to the alternative that may be more cost effective to construct. Several elements to consider are the amount of embankment and excavation required, bridge span lengths, angle of skew, sight distances, horizontal alignment, vertical alignment, constructability, traffic control, right of way, access, drainage, soil conditions, and overall construction costs.

If any crossings are planned for a future date, the mainline should preferably be designed as an underpass. Generally, future crossings over the mainline are easier to construct and are less disruptive to the mainline.

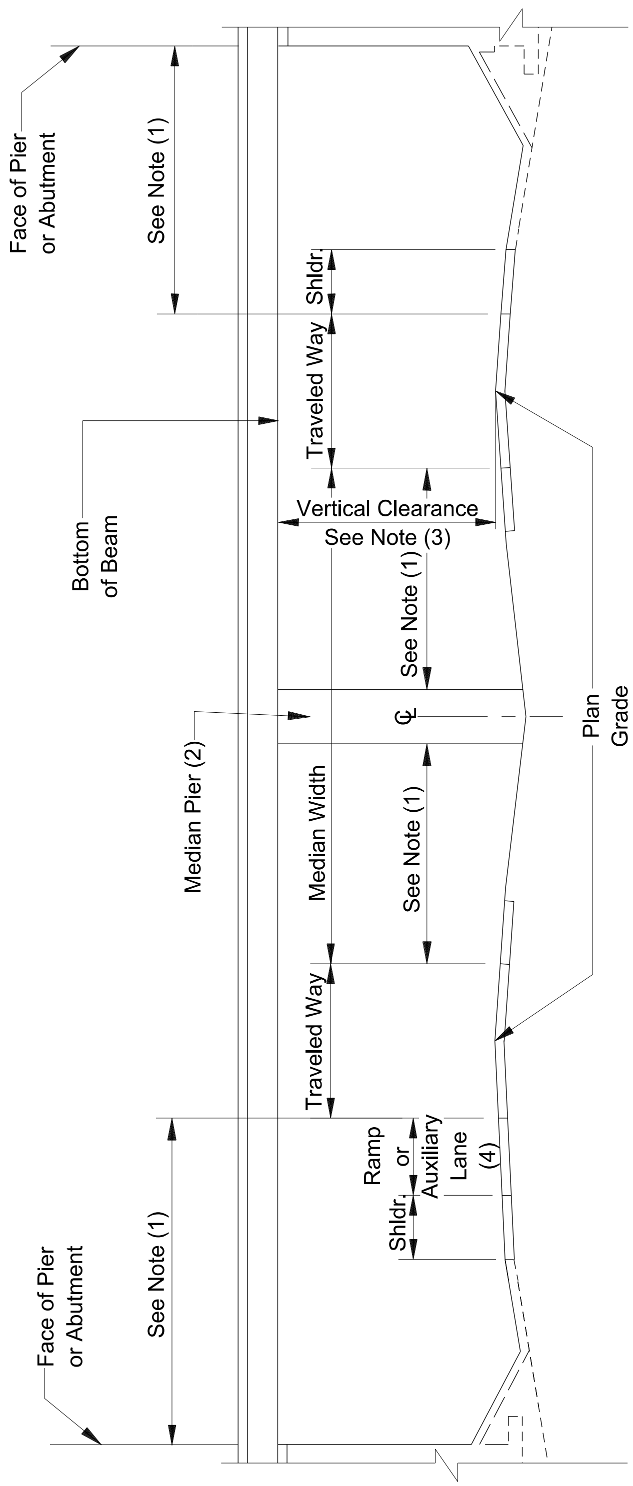
7-1.02 Underpasses

The design of the roadway at underpasses should be consistent with the design criteria of the rest of the roadway to the extent feasible. Cross-section elements such as the traveled way, shoulders, median, sidewalks, clear zones, etc. should preferably be maintained through the separation. Reductions to any cross-section elements that are necessary due to constraints related to an existing underpass may require approval by the Chief Engineer. See Section 2-10.0.

For underpasses along divided multilane roadways with open shoulders and a depressed median, the lateral clearances from the edge of the traveled way to the face of the pier or abutment should be a minimum distance of 30 feet (See Figure 7-1-A). If the lateral clearance is less than 30 feet, an appropriate barrier should be used to shield the pier or abutment wall (See Figure 7-1-B). In this situation, the minimum lateral clearance should be equal to the shoulder width and should be measured to the base of the barrier.

The lateral clearances on the inside of each roadway of a divided multilane roadway are usually governed by the median width and clear zone. If the roadways are separated by a concrete median barrier (CMB), the minimum median width should be equal to the widths of the two inside shoulders plus the width of the continuous median barrier (CMB), which is typically two feet wide.

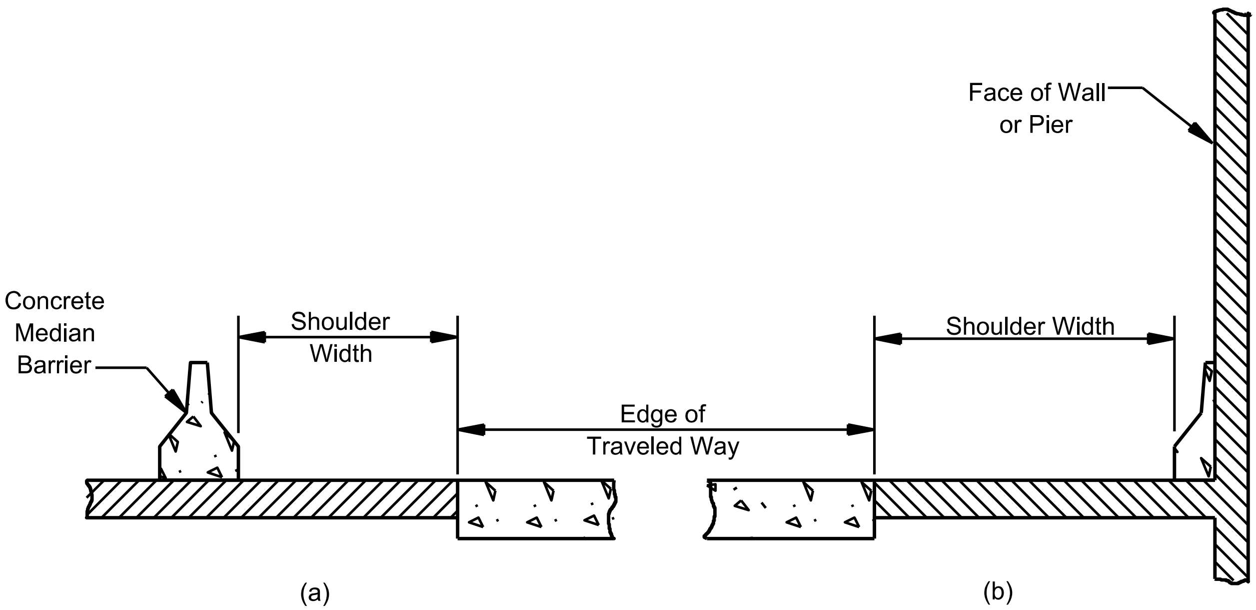
For a 2-lane roadway, an undivided multilane roadway, or a divided multilane roadway with curb and gutter, the width of the cross section at underpasses may vary depending on the design criteria appropriate for the particular functional classification and traffic volume.



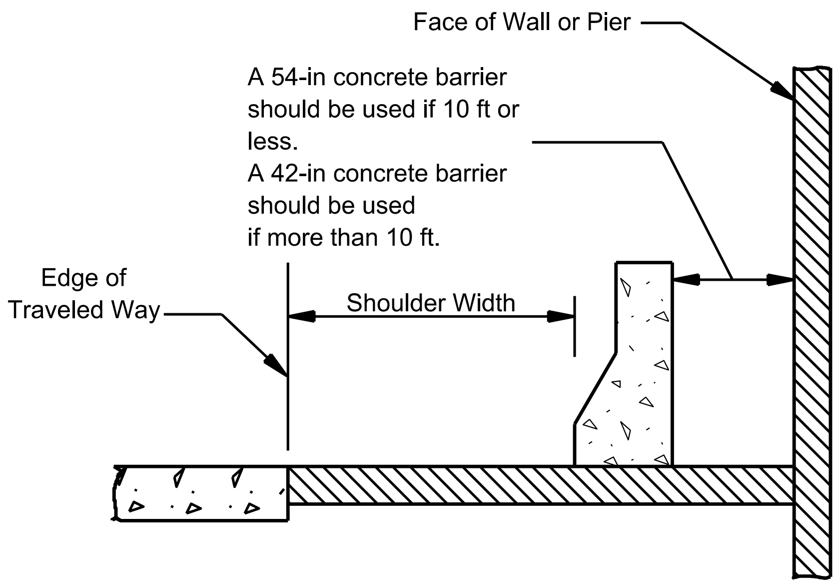
Notes:

1. *Recommended clear zone should be provided, or the pier or abutment should be protected with an appropriate barrier. See Chapter 9, Roadside Safety."*
2. *Median piers should typically be located on the median centerline. When the median width provides less than the recommended clear zone width, see Note (1) above.*
3. *For clear zones, an auxiliary lane greater than 0.5 miles is considered part of the traveled way.*

CLEARANCES FOR BRIDGES AT GRADE SEPARATIONS AND INTERCHANGES
Figure 7-1-A



Continuous Wall or Barrier



(c) With Barrier Right or Left

LATERAL CLEARANCES FOR MAINLINE UNDERPASSES
Figure 7-1-B

7-2.0 INTERCHANGES – PLANNING

7-2.01 Interchange Warrants

Although an interchange is a high-level alternative to an intersection, the high cost, proposed right of way, and environmental impacts of an interchange suggest that it should only be provided after costs and benefits have been carefully considered. When evaluating a proposed interchange, the following guidelines should be considered:

1. Functional Classification – Interchanges will be provided at all freeway-to-freeway crossings. Interchanges should be provided for all other crossings on roadways with fully-controlled access unless determined infeasible for some reason, such as interchange spacing. Interchanges for other roadways may be provided based on a case-by-case evaluation (e.g., costs, traffic volumes, alternative routes).

2. Capacity – An interchange may be considered where the Level of Service (LOS) at an at-grade intersection is undesirable and the intersection cannot be redesigned to operate at a desirable LOS. For example, when the volumes for an at-grade intersection exceed the capacity of the intersection, an interchange may be warranted.

3. Road-User Benefits – Interchanges may significantly reduce travel time and road user costs when compared to at-grade intersections by reducing delay, congestion, and crashes. Although an interchange may increase travel distance, the added cost of extra travel time is typically offset by the cost savings resulting from a reduction in delay. If an analysis reveals that road-user benefits over the service life of the interchange may exceed costs, then an interchange may be warranted.

4. Interchange Spacing – Minimum interchange spacing is provided in Table 7-2-A for urban and rural roadways. These distances are measured between the centerline of each crossroad. Greater separation of interchanges in transitions between urban areas and rural areas may be desirable.

Table 7-2-A
MINIMUM INTERCHANGE SPACING

Segment Location/Type	Spacing Minimums (miles)
Urban/Interstate	1*
Urban/Non-Interstate	1*
Rural/Interstate	3
Rural/Non-Interstate	2

* *In urban areas, spacing of less than one mile may be developed with grade-separated ramps or with Collector-Distributor (C-D) roadways if a capacity analysis results in a desirable LOS. See Section 7-2.10 for information on C-D roadways.*

7-2.02 Interchange Types

Interchanges should provide for all movements, even when the anticipated turning volume is low. An omitted maneuver may be a point of confusion to those drivers searching for the exit ramp or entrance ramp. In addition, unanticipated future developments may increase the demand for that maneuver.

The following factors should be evaluated when selecting an interchange type:

1. compatibility with the roadway system and functional classification of the crossroad
2. route continuity
3. LOS for each interchange element (e.g., ramp junction, ramp proper, ramp terminal)
4. operational and safety considerations (e.g., signing)
5. availability of access control along the crossroad
6. road-user impacts (e.g., travel distance and time, convenience, comfort)
7. driver expectancy
8. geometric design
9. right of way impacts and availability
10. construction and maintenance costs
11. constructability and maintenance of traffic
12. accommodation of pedestrians and bicyclists on crossroad
13. environmental impacts
14. potential growth of surrounding area

An interchange may be a modification of one of the basic types or a combination of two or more basic types. The basic interchange types used in Mississippi are:

1. Diamond – The diamond interchange is the simplest and most common type of interchange. Diagonal ramps should be provided in each quadrant with two at-grade intersections provided at the crossroad. Where left-turning movements are low to and from the mainline, diamond interchanges are often an appropriate alternative. The capacity of diamond interchanges is limited by the capacity of the ramp/crossroad intersection, also known as the ramp terminal (see Section 7-3.05). As left-turn movements at diamond interchanges increase, various operational improvements (e.g., additional lane(s), traffic signal(s), roundabout(s)) at one or both ramp terminals may be warranted.

2. Full Cloverleaf with Collector-Distributor (C-D) Roadways – Cloverleaf interchanges include loop ramps in all four quadrants to accommodate left-turn movements. Full cloverleaf interchanges with C-D roadways are the minimum type interchange used at the intersection of two roadways with full access control due to the weaving that results from adjoining loop ramps. C-D roadways allow the weaving to be transferred from the mainline, thereby avoiding any reduction of speed of the through traffic. Full cloverleaf interchanges with C-D roadways require a substantial amount of right of way; therefore, they are more common in suburban and rural areas where more space is available. See Section 7-2.10 for more information on C-D roadways.

3. Partial Cloverleaf – Partial cloverleaf interchanges include loop ramps in one, two, or three quadrants and are an effective compromise between diamond and full cloverleaf interchanges. The usage of partial cloverleaf interchanges may be indicated by limited right

of way in one or more quadrants, a need to maintain continuous flow on left-turn movements that are disproportionately high, or operational limitations of the crossroad that warrant the elimination of certain crossing maneuvers. Where one or more left-turn volumes are significant (500 vehicles per hour or more), loop ramps may be added to allow for continuous flow. The arrangement of ramps should be such that any major turning movements are made by right-turn exits and entrances in order to prevent any delays for the through traffic. To avoid weaving sections on the mainline, loop ramps should desirably be in diagonal quadrants. C-D roadways should be used where loop ramps will be on the same side.

4. Other Interchange Types – Numerous other interchange types may be considered depending upon right of way availability, traffic volumes, etc. (See FHWA Report - *FHWA-HRT-09-060 Alternative Intersections/Interchanges Informational Report* as well as ITE's *Freeway and Interchange Geometric Design Handbook* for additional information).

7-2.03 Basic Number of Lanes

The basic number of lanes is the minimum number of through lanes designated and maintained over a significant length of any roadway of arterial character. The basic number of lanes is based on the general volume level of traffic over that significant length; therefore, the number of lanes should remain constant over a significant distance regardless of localized changes in traffic volume and lane balance needs.

A decrease or an increase in the basic number of lanes should be considered only when there is a reduction or an increase in the traffic volume over a substantial length of the roadway. An increase in the traffic volume over short sections of roadway may be addressed by the addition of auxiliary lanes within those sections.

7-2.04 Lane Balance

Lane balance, which refers to balance in the number of travel lanes on the mainline and on the ramps, should be considered on high-volume roadways for efficient traffic operation through an interchange or series of interchanges. Figure 7-2-A illustrates how to coordinate lane balance and the basic number of lanes at interchanges. After the basic number of lanes is determined, the balance in the number of lanes should be checked considering the following principles:

- Exits – The number of approach lanes to the mainline exit (N_C) should equal the sum of the number of mainline lanes beyond the exit (N_F) plus the number of exiting lanes (N_E) minus one. Exceptions to this principle would be at cloverleaf loop ramp exits that follow a loop ramp entrance, or at exits between closely spaced interchanges (e.g., interchanges where the distance between the taper end of the entrance ramp junction and the beginning taper of the exit ramp junction is less than 1500 feet, and a continuous auxiliary lane is used between the terminals). In these cases, the auxiliary lane may be dropped at a 1-lane exit, with the number of approach lanes (N_C) being equal to the number of through lanes beyond the exit (N_F) plus the exiting lane (N_E).

- Entrances – At entrances, the number of lanes beyond the merging of two traffic streams (N_C) should not be less than the sum of the approaching lanes (N_F+N_E) minus one, but may be equal to the sum of all traffic lanes on the merging roadway.

7-2.05 Lane Drops

Lane drops, where the basic number of lanes is reduced, should normally occur on the mainline away from any other turbulence (e.g., interchange exit ramps and entrance ramps). The number of travel lanes on the mainline should only be reduced one lane at a time. At exits, for example, dropping two mainline lanes at a 2-lane exit ramp violates the principles of lane balance. One lane should provide the option of remaining on the mainline. Lane balance also prohibits immediately merging both lanes of a 2-lane entrance ramp onto a mainline without the addition of at least one lane beyond the entrance ramp.

As discussed in Section 7-2.03, lane drops should only occur when there is a reduction in the traffic volume over a substantial length of the roadway. Figure 7-2-B illustrates the recommended design of a lane drop beyond an interchange. The following criteria should be considered when designing a lane drop:

1. Location

   a. Rural – A lane drop should occur approximately 2000 to 3000 feet beyond the end of the entrance ramp junction. This distance allows for adequate signing, pavement marking, and driver adjustments from the interchange. However, this distance is not so far downstream that drivers become accustomed to the number of lanes and are not surprised by the lane drop. A lane drop should preferably be located on a tangent horizontal alignment and, if applicable, on the approach side of a crest vertical curve.

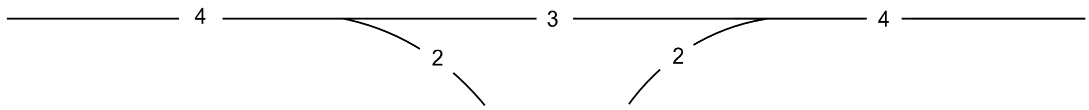
   b. Urban – Where interchanges are closely spaced, it may be necessary to drop a lane at an exit; however, a lane drop should not occur between closely spaced interchanges. The decision to drop a lane at an exit should be made on a case-by-case basis after an evaluation of operations of the traffic volume exiting versus the through traffic volume. Dropping the lane is preferable at a major divergence or 2-lane exit rather than at a 1-lane exit. Lane drops at exit ramps are further discussed in Section 7-2.04.

2. Sight Distance – Since lane drops can occur unexpectedly, especially to unfamiliar drivers, Decision Sight Distance (DSD) should be available to any point within the entire lane transition.

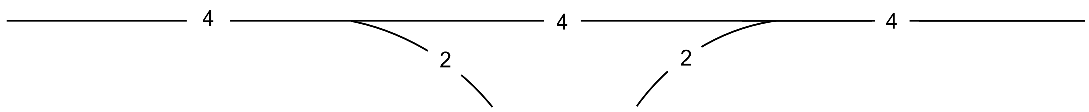
3. Outside Lane Drop – Outside lane drops are recommended due to the merging of slower vehicles and normal driver expectations. Where the left lane is to be continued in the median in the future, the outside lane drop is still recommended. In this case, the mainline should be designed for an outside lane drop and the traveled way shifted through a set of flat reverse curves. See Figure 7-2-C.

4. Shoulders – A minimum paved shoulder width of 10 feet, desirably 12 feet, should be provided for a distance of 350 feet beyond the lane drop for a recovery area for drivers.

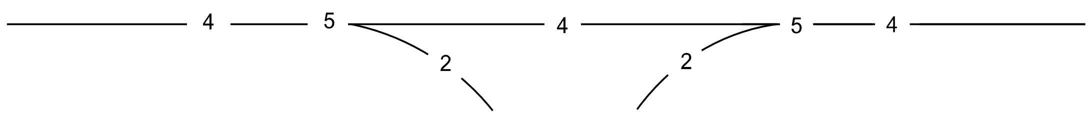
5. Transition – The transition for the lane drop should be similar to that of an entrance ramp; however, the taper rate should be longer than that of an entrance ramp. The desirable taper rate is 70:1 for the transition at the lane drop. The minimum taper rate is 50:1.



(a) Lane Balance but no Compliance with Basic Number of Lanes



(b) No Lane Balance but Compliance with Basic Number of Lanes



(c) Compliance with Both Lane Balance and Basic Number of Lanes

Exit: $N_C = N_F + N_E - 1$

Entrance:

Max: $N_C = N_F + N_E$

Min: $N_C = N_F + N_E - 1$

Where:

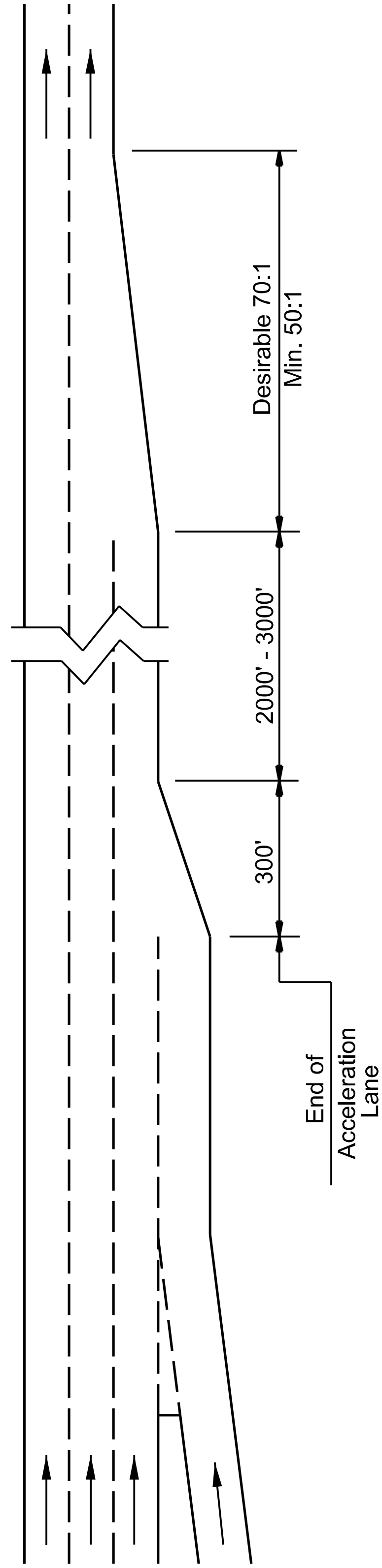
N_C = Number of Lanes for Combined Traffic

N_F = Number of Lanes on Freeway

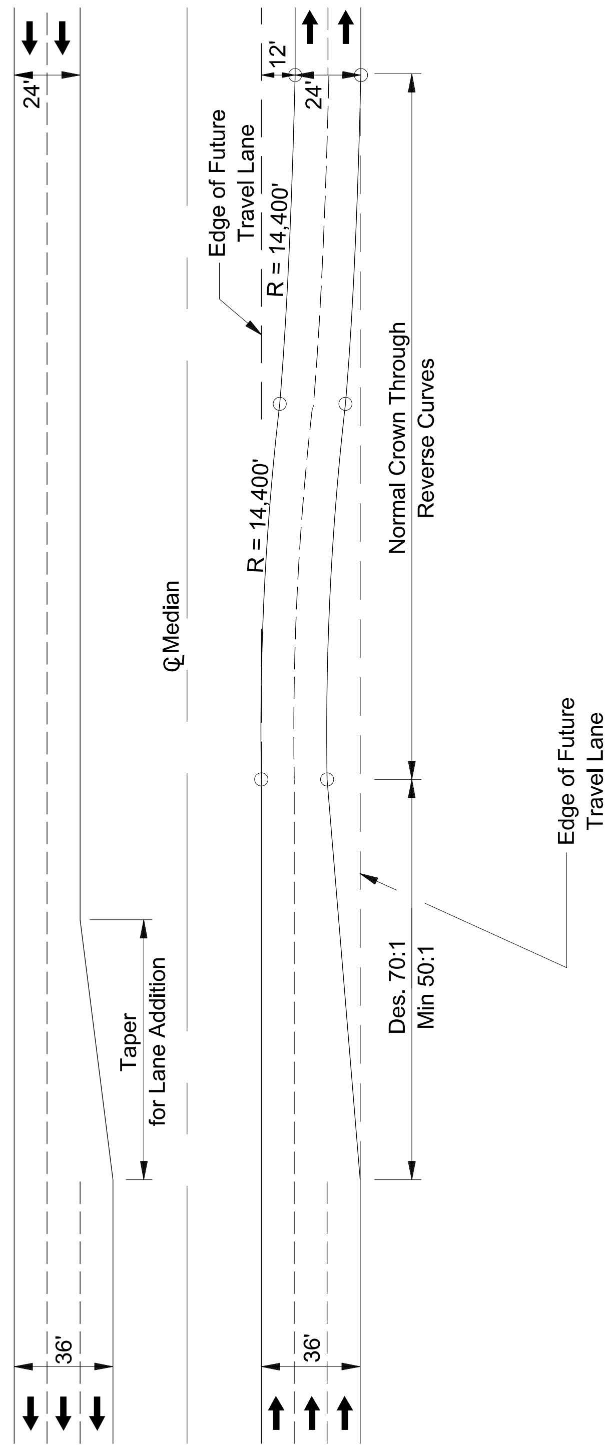
N_E = Number of Lanes on Exit or Entrance Ramp

(d) Lane Balance Equations

COORDINATION OF LANE BALANCE AND BASIC NUMBER OF LANES
Figure 7-2-A



TYPICAL FREEWAY LANE DROP (Outside)
Figure 7-2-B



TYPICAL FREEWAY LANE DROP (Inside)
Figure 7-2-C

7-2.06 Route Continuity

The mainline should flow continuously through an interchange. For routes that change direction, the driver should not be required to change lanes or exit to remain on the mainline. Route continuity without a change in the basic number of lanes is consistent with driver expectancy, simplifies signing, and reduces the decision demands on the driver.

7-2.07 Uniformity

To the extent feasible, all interchanges along a roadway should be uniform in geometric layout and general appearance. Except where unavoidable, all entrance ramps and exit ramps should be on the right side of the roadway. An inconsistent arrangement of exits between successive interchanges may cause driver confusion, resulting in drivers slowing down on high-speed lanes and making unexpected maneuvers.

7-2.08 Operational/Safety Considerations

Operations and safety are key considerations in interchange design; therefore, the following factors should be considered:

1. Exit Ramps

   a. Signing – Proper advance signing of exits is essential to allow necessary lane changes before the exit. The Traffic Engineering Division is responsible for determining the location of permanent signing.

   b. Deceleration – Sufficient distance should be provided to allow safe deceleration from the mainline design speed to the design speed of the first governing geometric feature on the ramp, typically a horizontal curve. See Section 7-3.04.01.1 for more information.

2. Entrance Ramps – Sufficient acceleration distance should be provided to allow a vehicle to attain a merge speed that is within five miles per hour of the design speed of the mainline at the point where the left edge of the entrance ramp joins the traveled way of the mainline. Where entrance ramps enter the mainline on an upgrade, the acceleration distance may be lengthened, or an auxiliary lane may be provided to allow vehicles to reach a safe speed prior to merging. See Section 7-3.04.02.1 for more information.

3. Driver Expectancy – An interchange should be designed to conform to the principles of driver expectancy, including route continuity, uniformity, lane balance, and spacing.

4. Roadside Safety – Because of the typical design features at interchanges, many fixed objects may be located within interchanges (e.g., signs at exit gores, bridge piers, rails). Objects that are located near decision points should be made breakaway, if feasible, or should be shielded with barriers or impact attenuators. See Chapter 9, “Roadside Safety”, for information on roadside safety.

5. Traffic-Controlled Ramp Terminals – Ramp terminals should have sufficient capacity so that the queuing traffic does not back up onto the mainline. In addition, sufficient access control should be maintained along the crossroad to allow the ramp terminal to work

properly. The vertical alignment should provide sight distance that meets or exceeds Stopping Sight Distance (SSD) to the ramp terminal traffic control devices. The *MUTCD* and *Access Management Manual* should be referenced for additional information.

6. Wrong-Way Maneuver – Channelized medians, islands, and/or adequate signing should be provided to minimize the potential for wrong-way movements.

7. Pedestrians and Bicyclists – Signing, crosswalks, barriers, grade separations, bridge sidewalks, and traffic control devices may be warranted to manage traffic movements and to control pedestrian and bicycle movements.

7-2.09 Auxiliary Lanes

As applied to interchange design, auxiliary lanes are most often used to comply with the principles of lane balance, increase capacity, and/or accommodate entering and exiting vehicles. The operational efficiency of the roadway may be improved if a continuous auxiliary lane is provided between entrance ramps and exit ramps of urban or suburban interchanges.

The following statements apply to the use of an auxiliary lane within or between interchanges:

- Within Interchange – Figure 7-2-D provides the basic schematics of alternative designs for adding and dropping auxiliary lanes within interchanges. The selected design may depend upon traffic volumes for the exiting, entering, and through movements.

- Between Interchanges – Where interchanges are closely spaced, an auxiliary lane should be provided where the distance between the end of the entrance taper of the first interchange and beginning of the exit taper for the second interchange is less than 1500 feet.

The design details for lane drops are provided in Section 7-2.05. Design details for exit ramp and entrance ramp junctions are provided in Section 7-3.04.

7-2.10 Collector-Distributor Roadways

A Collector-Distributor (C-D) roadway is an auxiliary roadway parallel to and separated from the mainline. A C-D roadway serves to collect and distribute traffic from multiple access points, thereby providing greater capacity and permitting higher operating speeds to be maintained on the mainline. C-D roadways are typically warranted where the speed differential between weaving and non-weaving vehicles is significant (see Section 7-2.12 for information on weaving sections). C-D roadways may be provided at single interchanges, through two adjacent interchanges, or continuously through several interchanges in urban areas. Figure 7-2-E illustrates a schematic of C-D roadways within a full cloverleaf interchange.

C-D roadways also allow the use of a single-exit approach when the interchange would otherwise consist of two exits, such as at full cloverleaf or partial cloverleaf interchanges. Usually, interchanges designed with one exit are preferred to those with two exits because the operational efficiency of the interchange may be improved. By using C-D roadways, deceleration and acceleration for exiting and entering vehicles is removed from the through lane. Additionally, the minimum DSD is easier to provide along the C-D roadway.

Therefore, C-D roadways should be used in conjunction with both full and partial cloverleaf interchanges. When designing C-D roadways, the following information should be considered:

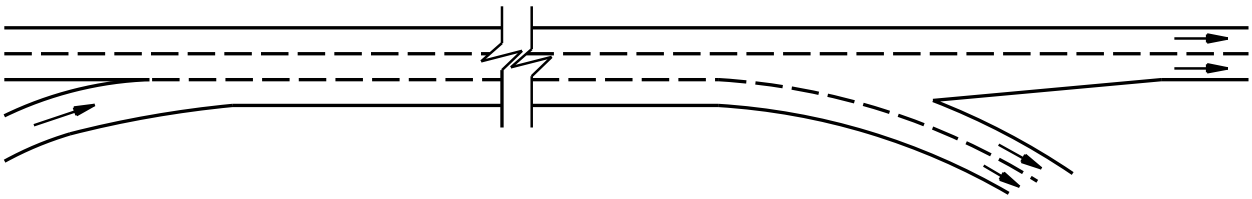
1. Design Speed – The design speed of a C-D roadway usually ranges from 40 to 65 miles per hour, typically within 10 miles per hour of the mainline design speed.

2. Lane Balance – Lane balance should be maintained at the exit and entrance points of the C-D roadways. See Section 7-2.04.

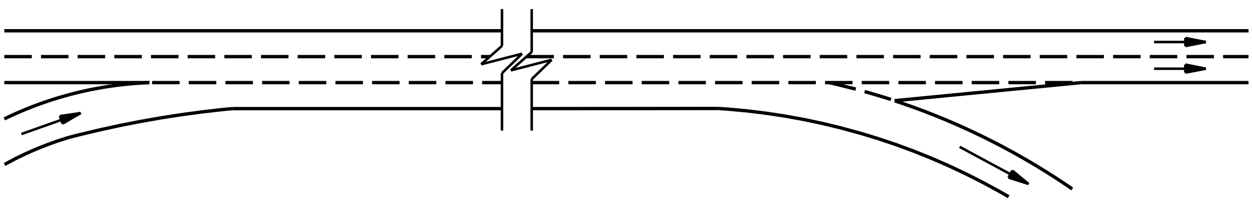
3. Width – C-D roadways may be one or two lanes, depending upon the traffic volumes and weaving conditions. C-D roadways are typically designed similar to ramps with traveled way widths of either 16 feet (one lane) or 24 feet (two lanes).

4. Separations – Shoulder widths, slopes, and clear zones for C-D roadways should be the same as the mainline in non-restricted areas. However, in restricted areas, the minimum width of separation should allow for shoulder widths equal to that on the mainline and for a suitable barrier to prevent indiscriminate crossovers.

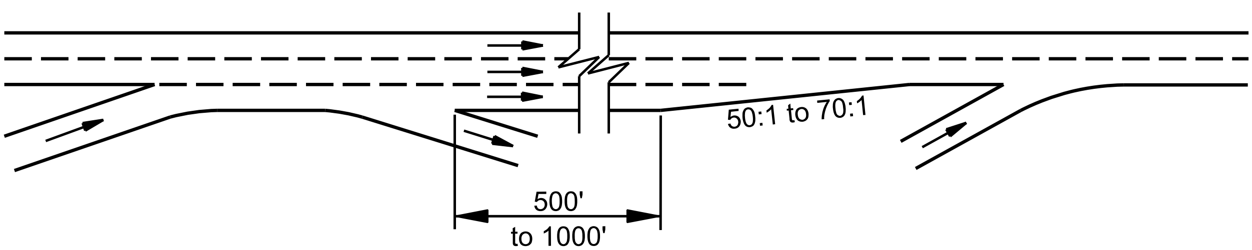
5. Ramp Junctions – Section 7-3.04 discusses the design criteria of ramp junctions. These criteria also apply to C-D roadway ramp junctions.



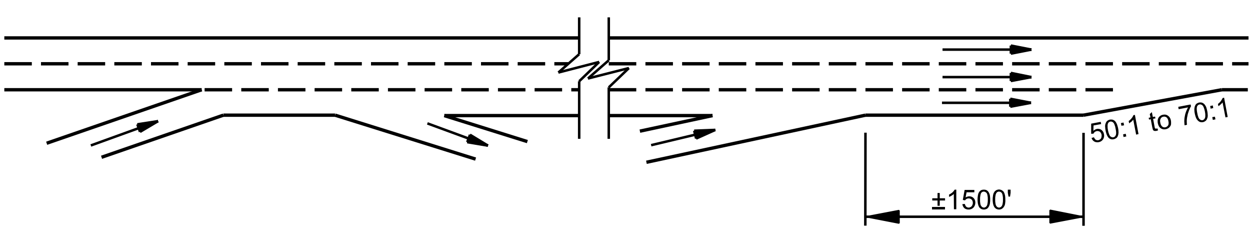
(a) Auxiliary Lane Dropped on Exit Ramp



(b) Auxiliary Lane Between Cloverleaf Loops or Closely Spaced Interchanges Dropped on Single Lane

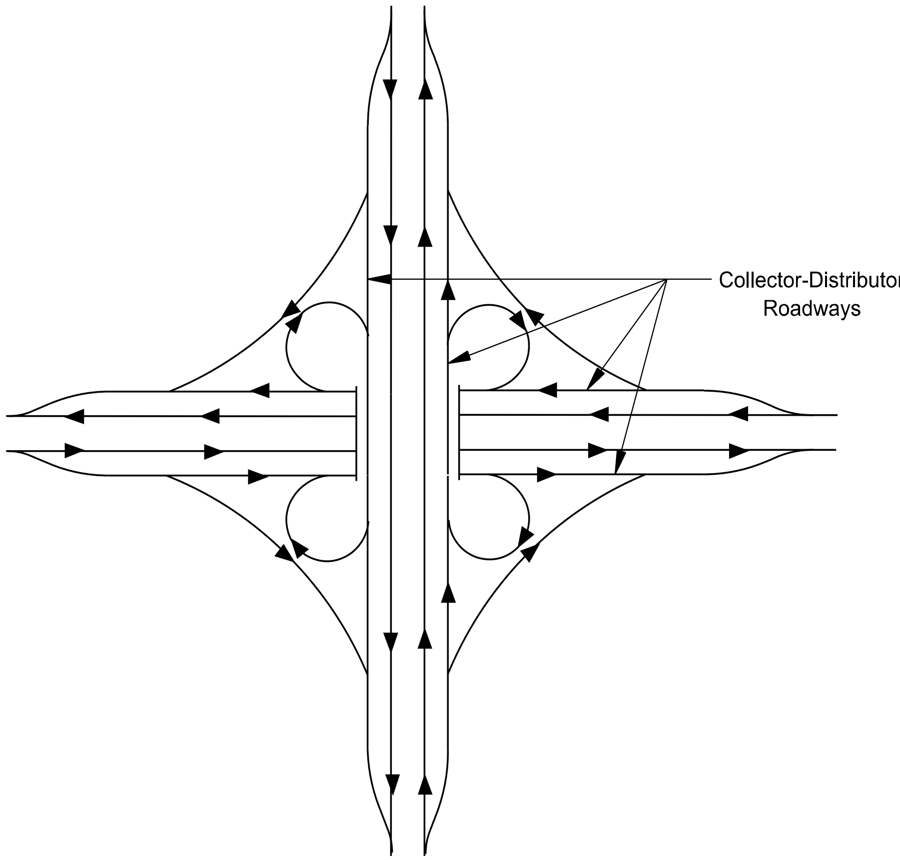


(c) Auxiliary Lane Dropped Within an Interchange



(d) Auxiliary Lane Dropped Beyond an Interchange

AUXILIARY LANES WITHIN AN INTERCHANGE
Figure 7-2-D



C-D ROADWAYS WITHIN CLOVERLEAF INTERCHANGES
Figure 7-2-E

7-2.11 <u>Signing and Marking</u>

Proper interchange operations depend on the compatibility between its geometric design and the traffic control devices at the interchange. The proper application of signs and pavement markings may increase safety, operational efficiency, and the clarity of paths to be followed. The logistics of signing along a roadway segment may also influence the minimum acceptable spacing between adjacent interchanges. The *Manual on Uniform Traffic Control Devices* (*MUTCD*) provides guidelines and criteria for the placement of traffic control devices at interchanges.

7-2.12 <u>Weaving Sections</u>

Weaving sections are segments where the pattern of traffic entering and exiting at adjacent points of access results in vehicular paths that cross each other. Weaving sections may occur within an interchange, or they may occur between entrance ramps of one interchange and exit ramps of a downstream interchange. The turbulent effect of weaving operations can result in reduced operating speeds and levels of service for through traffic. Weaving sections may be eliminated by using direct or semi-direct connections or by using C-D roadways.

The following information should be applied for weaving sections when C-D roadways are not an option:

- Weave Length – Weaving sections on a roadway should be the length determined using the *Highway Capacity Manual* at the desired LOS or a minimum of 1000 feet, whichever is greater.

- Level of Service – The weaving section should not have a significant adverse effect on the mainline in regard to LOS and safety; the desired LOS should be consistent with the acceptable LOS of the mainline and project area, which should be demonstrated through a systems analysis using the appropriate analysis tools.

7-2.13 Interchange Ramps

All interchanges consist of exit ramps and entrance ramps to provide access onto the mainline from a crossroad, or to provide access to a crossroad from the mainline. The following subsections provide information about the different types of ramps and design considerations for ramps, ramp junctions, and ramp terminals.

7-2.13.01 Ramp Types

Although ramps have varying shapes, each can be classified into one or more of the types illustrated in Figure 7-2-F and discussed in the following subsections.

7-2.13.01.1 *Diagonal Ramps*

Diagonal ramps, as shown in Figure 7-2-F(a), are a component of the diamond interchange and should include controlled ramp terminals. See Section 7-3.05 for more information on ramp terminal design.

7-2.13.01.2 *Loop Ramps*

- Free-Flow Loop Ramp – The free-flow loop ramp, as shown in Figure 7-2-F(b), consists of compounded circular arcs that turn through approximately 270 degrees. The free-flow loop ramp is a standard component of the cloverleaf interchange, four-quadrant partial cloverleaf interchange, and trumpet interchange.

- Controlled Loop Ramp – The controlled loop ramp, as shown in Figure 7-2-F(c), is a component of some partial cloverleaf interchanges. Controlled loop ramps should also preferably be designed with compound curves as described for free-flow loop ramps. Controlled terminals should be provided at intersections with crossroads that permit both right- and left-turning movements. See Section 7-3.05 for more information on ramp terminal design.

7-2.13.01.3 *Outer-Connector Ramps*

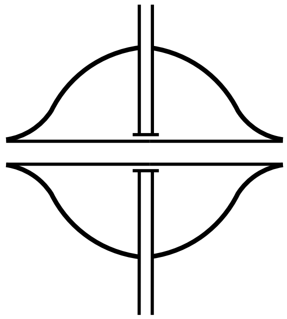
Outer-connector ramps, as shown in Figure 7-2-F(d), are in the same quadrant and to the outside of loop ramps. Outer-connector ramps may have free-flow operations (e.g., at cloverleaf or trumpet interchanges) or controlled operations (e.g., at partial cloverleaf interchanges).

7-2.13.01.4 *Semi-Direct Connection Ramps*

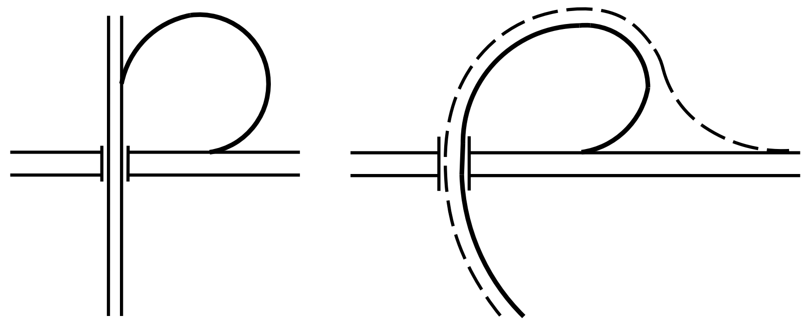
Semi-direct connection ramps, as shown in Figure 7-2-F(e), are indirect in alignment, yet more direct than a loop ramp. Drivers making a left turn normally exit to the right and reverse direction before entering the intersecting roadway. The outer connection of the trumpet interchange is also a semi-direct connection.

7-2.13.01.5 *Direct Connection Ramps*

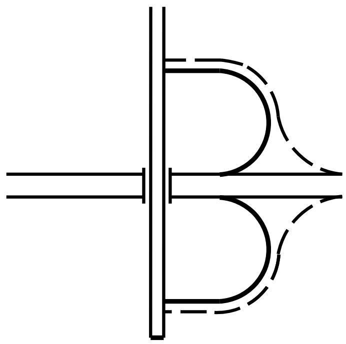
Direct connection ramps, as shown in Figure 7-2-F(f), are a component of a trumpet interchange. This type of ramp does not deviate greatly from the intended direction of travel and should be used to accommodate right-turning traffic on cloverleaf and directional interchanges.



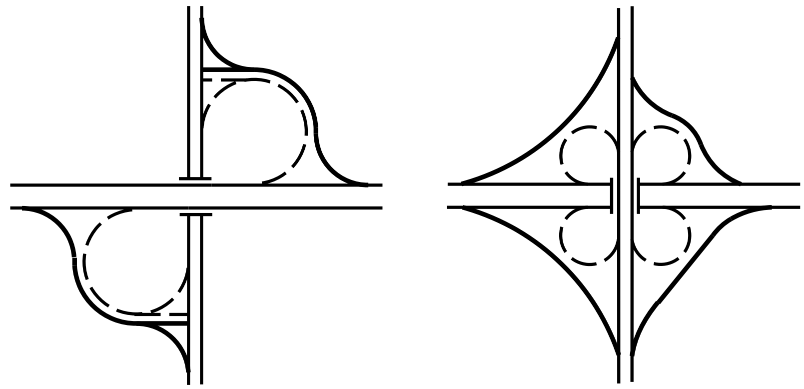
(a) Diagonal Ramps



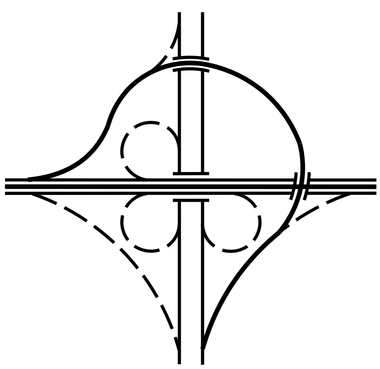
(b) Free-Flow Loop Ramps



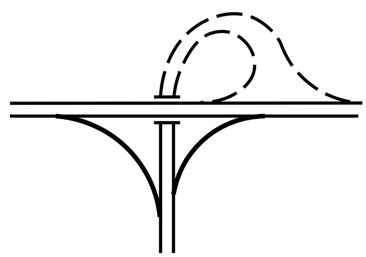
(c) Controlled-Loop Ramps



(d) Outer-Connector Ramps



(e) Semidirect Connection



(f) Direct Connection

Note: The heavier solid line indicates the ramp type being addressed.

RAMP TYPES
Figure 7-2-F

7-3.0 INTERCHANGES – DESIGN

7-3.01 Grading

The grading of an interchange should be considered early in the design process. Factors that should be considered include, but are not limited to:

1. Each interchange preferably being designed to fit the individual site conditions.

2. The cost of grading and structure(s) resulting from one roadway going over the other is a major factor in determining if the mainline should be an overpass or an underpass.

3. Construction sequences factoring into the decision. For example, the crossroad at a mainline underpass requiring a detour road in order to maintain traffic while the crossroad is being re-graded.

4. A mainline underpass providing an advantage to approaching drivers as the structure of the crossroad gives a visual indication of the upcoming interchange.

5. Ramp grades for a mainline underpass assisting drivers in decelerating as they exit the mainline and in accelerating as they approach the mainline.

7-3.02 General Considerations

1. Design Year – Typically, a 20-year design period from the anticipated opening date of the roadway should be used.

2. Crossroad Design Speed – The crossroad design speed should be based on its functional classification and urban or rural classification. See the geometric design criteria tables in Chapters 2, 12, and 14.

3. Horizontal Alignment – Horizontal alignments for both roadways through the interchange should desirably be tangent. Where this is not feasible, the following guidelines should be considered:

    a. Mainline Curving to the Left – Exit ramps should desirably not be placed in a location that would give the appearance of a continuing mainline tangent as the mainline curves to the left. In such cases, a parallel exit ramp with the taper beginning in the tangent section of the mainline is preferred. To make the deceleration lane more apparent to approaching drivers, a shorter taper (no more than 100 feet in length) should preferably be used. See Section 7-3.04.01 for more information on exit ramps.

    b. Superelevation – The horizontal alignment should preferably be designed so that superelevation, including transitions, are not proposed through ramp junctions or ramp terminals. Where a curve cannot be avoided, a curve with above-minimum radii should desirably be provided so that the superelevation does not result in a cross-slope rollover that exceeds the maximum amount shown in Table 6-7-D.

c. Sight Distance – Horizontal curves through interchanges have the potential to restrict sight distance if the minimum radius for the curve and the minimum lateral clearance to bridge piers, abutments, bridge rails, etc. are used. Therefore, above-minimum radii should be used if a tangent alignment cannot be provided. If sufficiently flat curvature cannot be provided, the lateral clearance should be increased in order to provide sufficient sight distance, even though this may result in increased bridge length and/or width.

4. Vertical Alignment – Vertical alignments for both roadways through the interchange should desirably be as flat as feasible for high visibility. Where this is not feasible, preference should be given to the mainline. Other design considerations for vertical alignment include:

   a. Ramps – Exit ramps should depart from the mainline where no crest vertical curvature restricts visibility of the ramp taper. If this situation cannot be avoided, the exit ramp should be extended with a parallel lane that begins before the crest of the vertical curve. See Section 7-3.04.01 for more information.

   b. Crossroad – Flat approach grades with proper drainage design should desirably be provided along the ramp terminals adjacent to the crossroad. For additional information on storage platforms at ramp terminals, see Section 6-1.04.

   c. Sight Distance – Although vertical alignments are typically controlled by SSD, Intersection Sight Distance (ISD) will typically control the vertical alignment of the crossroad through the interchange. For more information on ISD, see Section 6-6.0. Sight distance along the ramp proper should meet or exceed SSD for the ramp design speed.

5. Ramp Terminal – Once Items 1 through 4 above have been addressed, the best intersection of the mainline and the crossroad should be determined. Right of way and environmental impacts, minimum length of access control along the crossroad, and topography are factors that may influence this determination. See Section 7-3.05 for more information on sight distance at ramp terminals.

7-3.03 Ramp Design

7-3.03.01 Design Speed

Table 7-3-A provides the recommended ranges of ramp design speeds based on the design speed of the mainline. In addition, the following information should be considered when selecting the ramp design speed:

1. Loop Ramps – Design speeds in the upper ranges are generally not attainable for loop ramps.

   a. For rural loop ramps, the minimum design speed should be 30 miles per hour. See Section 7-2.13.01.2 for more information concerning the preferred design using compound curves. For loop ramps designed with compound curves, the minimum design speed only applies to the central curves, while the initial and final curves should have a design speed between that of the central curve and the adjacent mainline.

b. For loop ramps on C-D roadways, loop ramps in restricted urban conditions, or where mainline design speeds are 50 miles per hour or below, the minimum design speed for loop ramps should be 25 miles per hour.

c. Where truck traffic is greater than 15% of the total ADT, the minimum design speed should be 30 miles per hour.

d. Where less-restrictive conditions exist, the design speed of the loop ramp should preferably be increased.

2. Outer Connector Ramps – The design speed for the outer connector ramp of a rural cloverleaf interchange should typically be 50 miles per hour. Where a portion of the outer connector ramp parallels an adjacent loop ramp, a minimum design speed of 30 miles per hour should be used for the central curve.

3. Semi-Direct Connections – Design speeds in the middle to upper ranges should be used for semi-direct connections; however, the design speed should not be less than 30 miles per hour.

4. Direct Connections – A design speed in the middle to upper ranges should be used; however, the design speed should not be less than 40 miles per hour.

5. Controlled Terminals – If a ramp is terminated at an intersection with stop or signal control, design speeds in Table 7-3-A are not applicable to the portion of the ramp near the intersection.

Table 7-3-A
RAMP DESIGN SPEEDS

	Mainline Design Speed (mph)				
	50	**55**	**60**	**65**	**70**
	Ramp Design Speed (mph)				
Upper Range	45	50	50	55	60
Middle Range	35	40	45	45	50
Lower Range	25	30	30	30	30/35*

* *30 miles per hour for loop ramps – 35 miles per hour for all other ramps.*

Table 7-3-B presents geometric design criteria for interchange ramps based on the selected design speed (e.g., sight distance, horizontal alignment, vertical alignment). These criteria are discussed in the following subsections.

7-3.03.02 Cross-Section Elements

The following criteria apply to the ramp cross section:

1. Width – The standard ramp lane width is 16 feet; however, a 20-foot width should be used for loop ramps. For multilane ramps, each lane width should be the same as the mainline lane widths.

2. Cross slope (tangent sections) –

    a. The ramp traveled way should be sloped unidirectionally at 2% towards the outside shoulder.

    b. Shoulder cross slopes should typically be 4%.

    c. The inside shoulder should be sloped away from the ramp traveled way.

3. Shoulders – The standard paved shoulder widths are eight feet for the outside shoulder and three feet for the inside shoulder. Wider right shoulders on the mainline should be transitioned when there are narrower shoulders on the ramp (e.g., from 10 feet paved to eight feet paved). The shoulder width should be transitioned as shown in the *Standard Drawings.* For multilane ramps, the inside shoulder width should desirably be the same as the outside shoulder width.

4. Curbs – Curbs should not be used on ramps with design speeds greater than 40 miles per hour.

5. Side Slopes/Ditches – Side slopes and ditches on ramps should meet the same criteria as on the mainline.

6. Roadside Safety – The clear zone should be measured from the edge of the traveled way on both sides of the ramp using the criteria in Chapter 9, "Roadside Safety," which should be referenced for barrier warrants, selection, and layout criteria.

7-3.03.03 Horizontal Alignment

The following information applies to the horizontal alignment of ramps:

1. Minimum Curve Radii – Table 7-3-B provides minimum curve radii based on ramp design speed and an $e_{max} = 10\%$.

2. Curve Type – On all ramps except loop ramps, simple curves should be used unless field constraints dictate the use of compound curvature.

3. Loop Ramps –

    a. Minimum Curve Radii – The applicable e_{max} table, the selected design speed, and the desired design superelevation (e_d) should be used to determine the minimum radius for loop ramps. As discussed in Section 7-2.13.01.2, loop ramps should preferably be designed with compound curves, such that the central curve has a sharper radius than that of the initial and final curves. The flatter initial and final curves allow drivers to decelerate from the speed of the mainline over the initial portion of the ramp and to accelerate uniformly over the final portion of the ramp. The ratio of the flatter radius to the sharper radius should be 1.5:1 when feasible, but the ratio should preferably not exceed 2:1. Where less-restrictive conditions exist, the curve radii may be increased.

b. Superelevation – The maximum superelevation (e_{max}) rate of 10% should typically be used for horizontal curves on loop ramps. See Section 3-4.0.

c. Deceleration Distance – A deceleration lane should be provided for the loop exit ramp that allows a sufficient distance for a vehicle to decelerate from the design speed of the mainline to the design speed of the first controlling design element of the loop ramp, such as the horizontal curve near the exit gore. See Section 7-3.04.01.1.

d. Acceleration Distance – An acceleration lane should be provided for the loop entrance ramp that allows a sufficient distance for a vehicle to accelerate from the average running speed of the loop ramp curve to the average running speed of the mainline. See Section 7-3.04.02.1.

e. Transition Length – The cross slope of deceleration/acceleration lanes for loop ramps should be transitioned to the superelevation rate for the horizontal curve at the PC/PT in accordance with the *Standard Drawings*.

4. Ramp Junctions – Superelevation for horizontal curves at the ramp junction is based on the principles of superelevation as discussed in Section 3-4.0 for mainline roadways. See the *Standard Drawings* for the recommended curve radius and superelevation rate for curves at high-speed exit ramp and entrance ramp junctions (V ≥ 50 miles per hour).

5. Baseline/Centerline

a. Ramp – Typically, the outside edge of the ramp traveled way should be used for horizontal control, vertical control, and the point of superelevation rotation.

b. Loop Ramp – Typically, the inside edge of the loop traveled way is used for horizontal and vertical control.

6. Sight Distance – Chapter 3, “Horizontal Alignment”, presents criteria for horizontal sight distance around horizontal curves based on curve radii and design speed. These criteria also apply to curves on ramps.

7. Controlled Ramp Termini – If horizontal curves on the ramps are near the intersection, the design speed and radius for the curve should be selected based on the expected operational speed on the curve. The superelevation of the curve may be controlled by the vertical alignment of the crossroad.

Table 7-3-B
RECOMMENDED ALIGNMENT CRITERIA FOR INTERCHANGE RAMPS

Geometric Design Criteria								
Ramp Design Speed (mph)	25	30	35	40	45	50	55	60
Stopping Sight Distance (ft)	155	200	250	305	360	425	495	570
Horizontal Alignment								
Minimum Radius (ft) e_{max} = 10%	126	200	292	410	540	694	877	1090
Minimum Length of Arc (ft)	See Table 7-3-C							
Vertical Alignment								
Maximum Grades (%)	5-7	5-7	4-6	4-6	3-5	3-5	3-5	3-5
Crest Vertical Curves (K-values)	12	19	29	44	61	84	114	151
Sag Vertical Curves (K-values)	26	37	49	64	79	96	115	136

Table 7-3-C
MINIMUM ARC LENGTHS FOR COMPOUND CURVES

Radius (ft)	**100**	**150**	**200**	**250**	**300**	**400**	**500 or more**
Minimum Length (ft)	60	70	90	120	140	180	200

Note: *These lengths are applicable to ramp curves followed by a curve 1/2 its radius or preceded by a curve of double its radius.*

7-3.03.04 Vertical Alignment

1. Grades – Maximum grades for the vertical alignment of ramps cannot be as definitively expressed as those for the mainline. General values of limiting gradient are shown in Table 7-3-B, but for any one ramp, the selected profile is dependent upon a number of factors, some of which are listed below:

   a. The flatter the gradient on the ramp, the longer the ramp may be. At restricted sites, it may be advantageous to provide a steeper grade to shorten the length of the ramp. However, sufficient acceleration or deceleration distance should still be provided.

   b. The steepest grades should be used for the central portion of the ramp. Grades for ramp junctions should be determined by the mainline grade, but storage platforms at intersections should be as flat as feasible. See Section 6-1.04 for more guidance concerning profiles at intersections.

c. Short upgrades up to 5% do not unduly interfere with truck and bus operations. Consequently, the maximum gradient should desirably be limited to 5% for new construction.

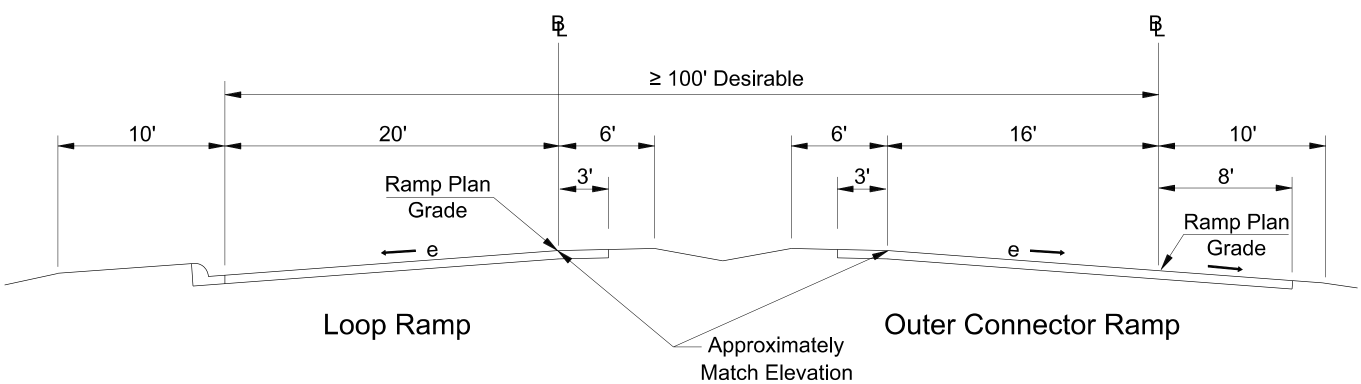
d. Downgrades on ramps should follow the same criteria as upgrades. However, where there are sharp horizontal curves, signed truck routes, heavy truck volumes, and/or bus traffic, downgrades should be limited to 4%.

e. The ramp grade within the ramp junction up to the physical nose should be approximately the same grade as that provided on the mainline. See Section 7-3.04 for more information regarding ramp junctions and gore areas.

2. Stopping Sight Distance – Sight distance along the interchange ramp, including vertical curves, should be designed to meet or exceed the minimum SSD criteria as presented in Chapter 4, "Vertical Alignment." Table 7-3-B provides K-values for crest and sag vertical curves on ramps. Where a vertical curve extends onto the ramp junction, the length of curve should be determined by using the mainline design speed.

7-3.03.05 Adjacent Ramps

Where two or more ramps are adjacent to each other (e.g. cloverleaf and trumpet interchanges), the horizontal and vertical alignments of each ramp should be designed such that the embankments do not conflict with each other. As shown in Figure 7-3-A, the profile of the baseline for the loop ramp and the inside edge of the travel lane of the outer connector ramp should be approximately parallel. The profile of the baseline for the outer connector ramp should then be calculated using the proposed width and superelevation. Once the ramps have diverged enough such that their embankments have adequate separation, their profiles can be set independently.



ADJACENT RAMP CROSS SECTION
Figure 7-3-A

7-3.04 Ramp Junctions

7-3.04.01 Exit Ramps

The Department uses two types of exit ramps — the taper type and the parallel-lane type. The taper type should desirably be used at all locations. Exceptions where the parallel-lane type may be used include, but are not limited to:

a. where the available deceleration distance is inadequate (see Section 7-3.04.01.1)
b. where all or part of the taper will cross a bridge
c. where a mainline horizontal curve to the left cannot be avoided

The *Standard Drawings* illustrate the design details for both the taper and parallel-lane exit ramps for single-lane exit ramps. The following subsections contain additional information for the design of exit ramps. For information on 2-lane exit ramps, see AASHTO's *A Policy on Geometric Design of Highways and Streets.*

7-3.04.01.1 *Deceleration Distance*

Sufficient deceleration distance should be provided to safely and comfortably allow an exiting vehicle to leave the mainline. The minimum length of deceleration is based upon the design speed of the mainline and the design speed of the first governing geometric control on the exit ramp, usually a horizontal curve. All deceleration should be able to occur within the full width of the exit ramp, beginning at the point at which the exit taper reaches a 12-foot width.

Table 7-3-D contains the minimum length of deceleration for various ramp and mainline design speeds. However, each standard drawing is based upon the length of deceleration needed for a 50 mile per hour ramp design speed and a 70 mile per hour mainline design speed. The following additional information should be evaluated for each specific interchange:

1. Additional Deceleration Length – In some cases, the standard drawing may not provide the minimum deceleration length for the proposed design speeds as shown in the table. Therefore, longer deceleration distances, as shown in Table 7-3-D, should be considered for exit ramps that warrant the additional distance. See Example 7-3-1.

2. Less Deceleration Length – In some cases, the standard drawing exceeds the values that are provided in Table 7-3-D. The standard drawing should still be used in these cases where no constraints exist (e.g., right of way and/or environmental impacts, bridge structures). However, the table may be used to determine an adequate deceleration distance where constraints exist that prohibit the exit ramp from being constructed in accordance with the standard drawing. In some cases where bridge structures prohibit the exit ramp from being designed according to the standard drawing, extending the ramp across the entire structure may be considered. See Example 7-3-2.

3. Adjustment for Grade – If the deceleration area of an exit ramp is on a downgrade of 3% or more, an adjustment in the deceleration distance may be necessary in accordance with the criteria in Table 7-3-E. The adjustment factor shown in Table 7-3-E should be applied to the minimum length shown in Table 7-3-D and then compared to the standard drawing. If the adjusted value exceeds the distance in the standard drawing, the exit ramp should be designed to provide the adjusted deceleration distance. No adjustment is necessary,

however, to decrease the length of the ramp if the deceleration area is on an upgrade of 3% or more unless constraints exist. See Example 7-3-3.

4. Projects on Existing Roadways – For projects on existing roadways where 3R criteria is being used, Table 7-3-D and Table 7-3-E should be used to evaluate existing interchanges within the limits of a project to determine if the existing entrance ramps provide adequate deceleration length. If the existing deceleration length does not meet the distance provided in the table for the appropriate design speeds, the ramp should desirably be extended to provide this distance. However, providing the entire length shown in the table (for those that exceed 340 feet) may be cost-prohibitive, particularly at interchanges with constraints and/or those that do not have a crash pattern associated with drivers decelerating in the through lane as they prepare to exit the mainline. In such cases, the exit ramp should be extended in accordance with the standard drawing.

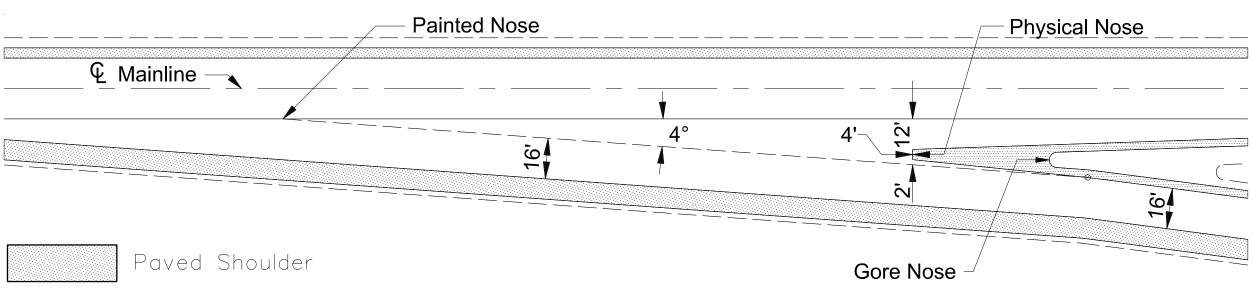
When interchanges are designed such that the exit ramp is not consistent with the deceleration distance shown in the standard drawing, the roadway design plans should include a drawing that illustrates the modified details of the exit ramp.

7-3.04.01.2 *Decision Sight Distance*

There should be a clear view of the entire ramp junction, including the exit nose and a section of the ramp roadway beyond the gore. However, vertical curvature can obstruct the exit points if not taken into consideration during design. Therefore, DSD should desirably be provided to the painted nose at the ramp junction of an exit ramp. See Section 4-5.02.02. If providing DSD is not feasible, the sight distance should exceed the SSD by 25% (i.e., 1.25 × SSD) and should be available throughout the ramp junction. The desirable object height is 0.0 feet (i.e., the roadway surface); however, an object height of 2.0 feet may be used.

7-3.04.01.3 *Exit Gore Area*

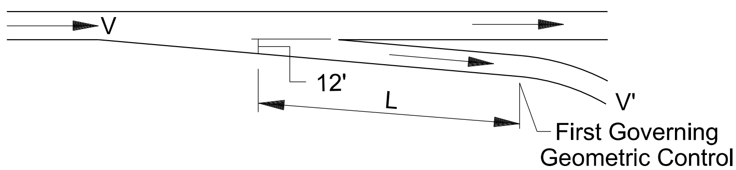
The exit gore area is normally considered to include the graded area that may extend downstream beyond the gore nose plus the paved triangular area between the mainline and the exit ramp. Figure 7-3-B illustrates the locations of the painted nose, physical nose, and gore nose at a typical exit ramp.



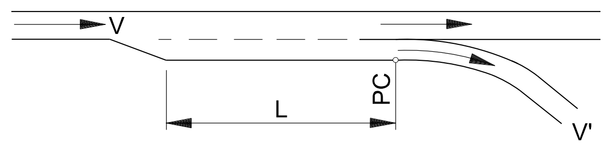
EXIT GORE ELEMENTS
Figure 7-3-B

Table 7-3-D
MINIMUM LENGTH OF DECELERATION

Mainline Design Speed, (mph) (V)	L = Deceleration Length (ft)					
	For Design Speed of Exit Curve, V′ (mph)					
	25	**30**	**35**	**40**	**45**	**50**
30	140	-	-	-	-	-
40	235	185	155	-	-	-
50	355	315	285	225	175	-
60	460	430	405	350	300	240
65	500	470	440	390	340	280
70	550	520	490	440	390	340



Taper Exit Ramp



Parallel Exit Ramp

Notes:

1. *The deceleration lengths are calculated from the distance for a passenger car to decelerate from the design speed of the mainline to the design speed of the first governing geometric control.*

2. *These values are for grades < 3%. For downgrades ≥ 3%, these values should be adjusted in accordance with the grade adjustment factors provided in Table 7-3-E.*

Table 7-3-E
GRADE ADJUSTMENTS FOR DECELERATION

Direction of Grade	Ratio of Deceleration Length on Grade to Length on Level		
	< 3%	3% ≤ G < 4%	5% ≤ G < 6%
Upgrade	1.0	0.9	0.8
Downgrade	1.0	1.2	1.35

Notes:

1. *The table applies to all mainline design speeds.*
2. *The grade in the table is the average grade along the distance for which the acceleration length applies.*

* * * * * * * * * *

Example 7-3-1

Given: Mainline Design Speed = 65 miles per hour
First Exit Curve Design Speed = 40 miles per hour
Average Grade = 1% downgrade

Problem: Determine if the deceleration length in the standard drawing is adequate.

Solution: Table 7-3-D yields a minimum deceleration distance of 390 feet for a grade that is less than 3%. With the average grade being less than a 3% downgrade, no adjustment factor for grade is needed.

The table value exceeds the distance shown in the standard drawing. Therefore, the deceleration distance should be increased to a minimum length of 390 feet.

Example 7-3-2

Given: Mainline Design Speed = 60 miles per hour
First Exit Curve Design Speed = 45 miles per hour
Average Grade = 1% downgrade

Problem: Determine the minimum length of deceleration.

Solution: Table 7-3-D yields a minimum deceleration distance of 300 feet for a grade that is less than 3%. With the average grade being less than a 3% downgrade, no adjustment factor for grade is needed.

The standard drawing provides an adequate amount of deceleration length and should typically be used unless constraints exist.

<u>Example 7-3-3</u>

Given: Mainline Design Speed = 70 miles per hour
First Exit Curve Design Speed = 50 miles per hour
Average Grade = 4% downgrade

Problem: Determine the minimum length of deceleration.

Solution: Table 7-3-D yields a minimum deceleration distance of 340 feet for a grade that is less than 3%. According to Table 7-3-E, this distance should be increased by a factor of 1.2.

Therefore: L = 340 x 1.2
L = 408 feet

A minimum deceleration length of 408 feet should be provided from the full width of the exit ramp to the PC of the first exit curve. However, this length should preferably be rounded up to an even number, such as 450 feet.

* * * * * * * * * *

The following information should be considered when designing the exit gore:

1. <u>Obstacles</u> – The area beyond the gore nose should desirably be kept free of obstacles (excluding the ramp exit sign). Any obstacles within the clear recovery area of the gore nose should be made breakaway or shielded by a barrier or impact attenuator.

2. <u>Side Slopes</u> – The graded area along the ramp junction should be consistent with the adjacent mainline grade. However, at some sites, the vertical divergence of the ramp and mainline beyond the gore nose may warrant protection for both roadways. Non-traversable areas beyond the gore nose should be shielded by a barrier or impact attenuator. See Chapter 9, "Roadside Safety."

3. <u>Cross Slopes</u> – The paved triangular gore or neutral area between the through lane and the exit ramp should be safely traversable. See the *Standard Drawings* for proposed cross slopes in the gore area.

4. <u>Drainage</u> – Drainage inlets should normally be placed between the physical nose and the gore nose. The drainage design should avoid sheet flow across the travel lanes.

7-3.04.02 <u>Entrance Ramps</u>

Two ramp types have been used in the construction of entrance ramps — the taper type and the parallel-lane type. However, the Department uses the parallel-lane entrance ramp as its standard design for new and reconstructed interchanges. The *Standard Drawings* illustrate the design details for a typical single-lane entrance ramp. The following subsections contain additional information for the design of entrance ramps. For information on 2-lane entrance ramps, see AASHTO's *A Policy on Geometric Design of Highways and Streets.*

7-3.04.02.1 *Acceleration Distance*

Sufficient acceleration distance should be provided such that drivers may attain a merge speed that is within five miles per hour of the mainline design speed at the point where the left edge of the entrance ramp joins the mainline traveled way. The minimum length of acceleration is based upon the design speed of the last controlling horizontal curve on the entrance ramp and the design speed of the mainline. Acceleration lengths apply to the full width of the parallel lane; taper lengths are in addition to the minimum acceleration lengths.

Table 7-3-F contains the minimum length of acceleration for passenger cars for various ramp and mainline design speeds. However, the Department's standard drawing includes an entrance ramp with a 650-foot parallel acceleration lane to provide more time for merging vehicles to find a gap in the through traffic. The following additional information should be evaluated for each specific interchange:

1. Additional Acceleration Length – In some cases, the standard drawing may not provide the minimum acceleration length for the proposed design speeds as shown in the table. Therefore, longer acceleration distances as shown in Table 7-3-F should be considered for those entrance ramps that warrant the additional distance. See Example 7-3-4.

2. Less Acceleration Length – In some cases, the standard drawing exceeds the values that are provided in Table 7-3-F. The standard drawing should still typically be used in these cases where no constraints exist (e.g., right of way and/or environmental impacts, bridge structures). However, the table may be used to determine an adequate acceleration distance where constraints exist that prohibit the entrance ramp from being constructed in accordance with the standard drawing. See Example 7-3-5.

   In some cases where bridge structures prohibit the entrance ramp from being designed according to the standard drawing, extending the ramp across the entire structure may be considered.

3. Adjustments for Grade – If the acceleration area of an entrance ramp is on an upgrade of 3% or more, an adjustment in the acceleration distance may be necessary in accordance with the criteria in Table 7-3-G. The adjustment factor shown in Table 7-3-G should be applied to the minimum length shown in Table 7-3-F and then compared to the standard drawing. If the adjusted value exceeds the distance in the standard drawing, the entrance ramp should be designed to provide the adjusted acceleration distance. No adjustment is necessary, however, to decrease the length of the ramp if the acceleration area is on a downgrade of 3% or more unless constraints exist. See Example 7-3-6.

4. Capacity – Where the mainline and ramp are expected to carry traffic volumes approaching the design capacity of the merging area (as determined by a capacity analysis), greater acceleration lengths in addition to the distances shown in Table 7-3-F should be considered. Where capacity determines that additional length is warranted, the additional length should be calculated using the *Highway Capacity Manual*. Roadway Design Division should coordinate with the Planning Division in these cases.

5. Trucks – Where a capacity analysis indicates that trucks are expected to govern the design of the entrance ramp, the truck acceleration distances provided in Table 7-3-H should be

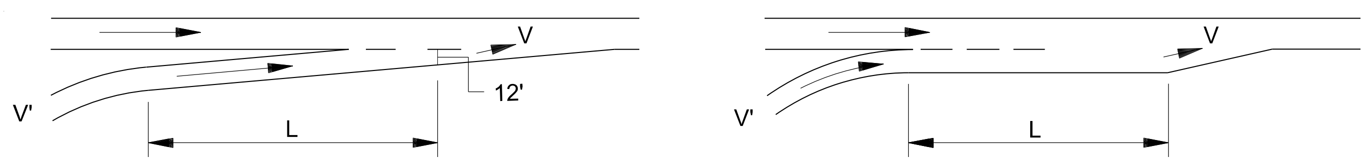
considered. Typical areas where trucks may govern the ramp design include weigh stations, truck stops, and transport staging terminals. Where upgrades exceed 3%, adjustments to the truck acceleration distance should be considered using the factors in Table 7-3-G.

6. Projects on Existing Roadways – For projects on existing roadways where 3R criteria is being used, Table 7-3-F and Table 7-3-G should be used to evaluate existing interchanges within the limits of a project to determine if the existing entrance ramps provide adequate acceleration length. If the existing acceleration length does not meet the distance provided in the table for the appropriate design speeds, the ramp should desirably be extended to provide this distance. However, providing the entire length shown in the table (for those that exceed 650 feet) may be cost-prohibitive, particularly at interchanges that do not have a crash pattern associated with the merge of the entrance ramp and the mainline. In such cases, the entrance ramp should be extended in accordance with the standard drawing.

When interchanges are designed such that the entrance ramp is not consistent with the standard acceleration distance of 650 feet, the roadway design plans should include a drawing that illustrates the modified details of the entrance ramp.

Table 7-3-F
MINIMUM LENGTH OF ACCELERATION
(Passenger Cars)

Mainline Design Speed (mph) (V)	L = Acceleration Length (ft)					
	For Design Speed of Entrance Curve (mph) (V')					
	25	30	35	40	45	50
30	-	-	-	-	-	-
40	210	120	-	-	-	-
50	550	450	350	130	-	-
60	1020	910	800	550	420	180
65	1220	1120	1000	770	600	370
70	1420	1350	1230	1000	820	580



Taper Entrance Ramp

Parallel Entrance Ramp

Notes:

1. *The acceleration lengths are calculated from the distance for a passenger car to accelerate from the design speed of the entrance curve to a speed that is within five miles per hour of the design speed of the mainline.*

2. *These values are for grades < 3%. For upgrades ≥ 3%, these values should be adjusted in accordance with the grade adjustment factors provided in Table 7-3-G.*

3. *The L distance or 300 feet should be provided beyond the painted nose, whichever is greater.*

Table 7-3-G
GRADE ADJUSTMENTS FOR ACCELERATION LANES

Mainline Design Speed (mph) (V)	Ratio of Length on Grade to Length for Design Speed of Entrance Ramp Curve (mph)				
	20	30	40	50	All Speeds
	3% to 4% upgrade				3% to 4% downgrade
40	1.3	1.3	-	-	0.7
50	1.3	1.4	1.4	-	0.7
60	1.4	1.5	1.5	1.6	0.6
65	1.5	1.6	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
	5% to 6% upgrade				5% to 6% downgrade
40	1.5	1.5	-	-	0.6
50	1.5	1.7	1.9	-	0.6
60	1.7	1.9	2.2	2.5	0.5
65	1.9	2.1	2.4	2.8	0.5
70	2.0	2.2	2.6	3.0	0.5

Notes:

1. *No adjustment is recommended for grades < 3%.*
2. *The grade in the table is the average grade along the distance for which the acceleration length applies.*

* * * * * * * * * *

Example 7-3-4

Given: Mainline design speed = 70 mph
Entrance ramp design speed = 45 mph
Average grade = 1% upgrade

Problem: Determine if the standard drawing length of 650 feet is adequate.

Solution: Table 7-3-F yields an acceleration length of 820 feet. With the average grade being less than a 3% upgrade, no adjustment factor for grade is needed.

The table value exceeds the 650-foot distance shown in the standard drawing; therefore, a minimum distance of 820 feet should be provided.

Example 7-3-5

Given: Mainline design speed = 60 mph
Entrance ramp design speed = 40 mph
Average grade = 2% upgrade
An existing bridge is located 900 feet from the beginning of the parallel acceleration lane shown in the standard drawing

Problem: Determine if the standard drawing length of 650 feet is necessary.

Solution: Table 7-3-F yields an acceleration length of 550 feet. With the average grade being less than a 3% upgrade, no adjustment factor for grade is needed.

With no constraints, an acceleration length of 650 feet and 300-foot taper should be used. However, in this example, the 950-foot distance required for acceleration and taper extends beyond the existing bridge abutment downstream. Therefore, a minimum acceleration length of 550 feet may be used along with a 300-foot taper.

Example 7-3-6

Given: Mainline design speed = 65 mph
Entrance ramp design speed = 50 mph
Average grade = 3% upgrade

Problem: Determine if the standard drawing length of 650 feet is adequate.

Solution: Table 7-3-F yields an acceleration length of 370 feet for a grade that is less than 3%. According to Table 7-3-G, this distance should be increased by a factor of 1.7.

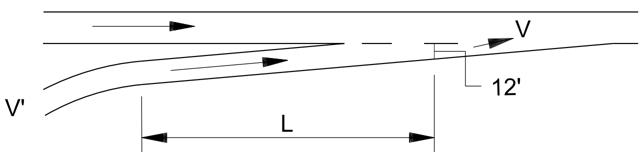
Therefore: L = 370 x 1.7
L = 629 feet

The standard drawing provides an adequate amount of acceleration length.

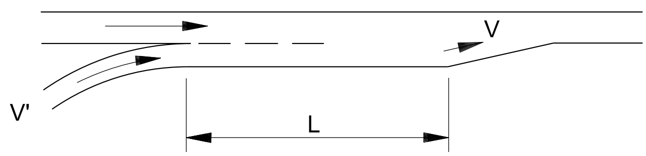
* * * * * * * * * *

Table 7-3-H
MINIMUM LENGTH OF ACCELERATION
(Trucks)

Mainline Design Speed (mph) (V)	L = Acceleration Length (ft)				
	For Design Speed of Entrance Curve (mph) (V′)				
	25	**30**	**35**	**40**	**50**
55	550	500	425	200	-
60	1150	1100	1025	800	525
65	1950	1900	1825	1600	1250
70	2650	2600	2525	2300	1850



Taper Entrance Ramp



Parallel Entrance Ramp

Notes:

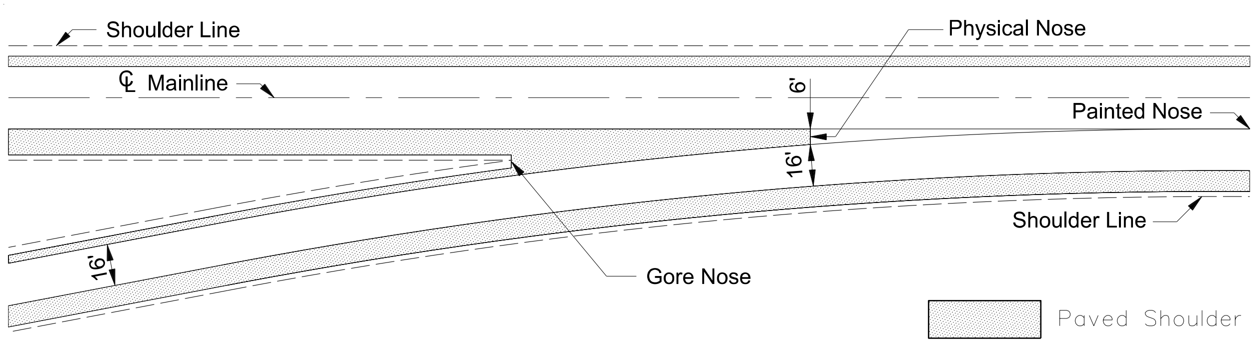
1. *For design speeds < 55 miles per hour, the L distance from Table 7-3-F should be used.*
2. *The acceleration lengths are calculated from the distance needed for a 200 LB per horsepower truck to accelerate from the design speed of the entrance curve to reach a speed that is within 10 miles per hour of the mainline design speed.*

7-3.04.02.2 *Sight Distance along Entrance Ramps*

Sight distance along entrance ramps should meet or exceed SSD for the design speed of the ramp. A clear view along the entire ramp proper, ramp gore area, and ramp junctions should be available. Where interchange quadrants are open-graded designs (with no retaining walls), the infield areas should be graded and left free of obstacles that interfere with direct sight of merging vehicles. In other words, drivers along entrance ramps should have direct sight of oncoming mainline traffic and, likewise, mainline drivers should have direct sight of oncoming ramp traffic.

7-3.04.02.3 *Entrance Gore Area*

The entrance gore area is normally considered to include the graded area in advance of the gore nose and the paved triangular area between the mainline and the entrance ramp. Figure 7-3-C illustrates the locations of the gore nose, physical nose, and painted nose at a typical entrance ramp.



ENTRANCE GORE CHARACTERISTICS
Figure 7-3-C

The following information should be considered when designing the entrance gore:

1. Obstacles – The gore area should be free of any signs, roadside barriers, side slopes, trees, plantings, etc., that may obstruct the sight distance of vehicles upstream of the ramp junction on the mainline and on the ramp.

2. Side Slopes – The graded area before the gore nose should be consistent with the adjacent mainline grade. However, at some sites, the elevation difference between the ramp and mainline may warrant protection for both roadways. See Chapter 9, "Roadside Safety."

3. Cross Slopes – The paved triangular gore or neutral area between the mainline and entrance ramp should be safely traversable. See the *Standard Drawings* for proposed cross slopes in the gore area.

4. Drainage – Drainage inlets should normally be placed between the physical nose and the gore nose. Drainage design should avoid sheet flow across the travel lanes.

7-3.05 Ramp Terminals

This section presents information that is applicable to the intersection of an interchange ramp and a crossroad. Additional applicable criteria for the at-grade intersections of ramp terminals and crossroads is located in Chapter 6, "At-Grade Intersections".

At most interchanges, the ramp will terminate or begin with an at-grade intersection (stop sign, traffic signal, or roundabout), which may involve a consideration of capacity and physical geometric design elements (e.g., sight distance, angle of intersection, acceleration lanes, channelization, turning lanes). The following information should also be applied in the design of the ramp terminal:

1. Crossroad Width (W) – The crossroad width may be based on anticipated traffic volumes for the design year, crossroad functional classification, and applicable design criteria.

2. Angle of Intersection – An exit ramp should intersect the crossroad at the ramp terminal as close to 90 degrees as feasible. This angle will allow the driver of the stopped vehicle to more easily look for a gap in both directions before entering traffic on the crossroad. The minimum angle of intersection, though, for an exit ramp is 75 degrees.

   At an entrance ramp, the angle of intersection has no impact on the driver's ability to find a gap in the opposing traffic to turn left from the crossroad onto the entrance ramp. Therefore, the angle of intersection for an entrance ramp does not need to meet the criteria for an exit ramp. However, the angle of intersection should be adequate to accommodate the design vehicle turning left from the crossroad onto the entrance ramp. For the driver turning right from the crossroad onto the entrance ramp, a skewed angle may provide some advantage to allow the driver to more easily look for a gap to merge into traffic that has turned left onto the entrance ramp.

3. Sight Distance – Section 6-6.0 discusses ISD for at-grade intersections, which also applies to ramp terminals. When designing an interchange, each ramp terminal should be located such that bridge piers, abutments, bridge rails, roadside barriers, etc. do not present sight obstructions for drivers stopped at the ramp terminal as they look for a gap to enter the crossroad. Additionally, the vertical alignment will typically be controlled by ISD rather than SSD so that adequate sight distance is provided. When calculating ISD, consideration should be given to any future widening of the crossroad that may affect the vertical alignment of the ramp terminal.

4. Capacity – In urban areas where traffic volumes are often high, inadequate capacity of the ramp terminal can adversely affect the operation of the ramp junction. In a worst-case situation, a backup onto the mainline may impair the safety and operations of the mainline itself; therefore, additional consideration should be given to providing sufficient capacity and storage for an at-grade intersection or a merge with the crossroad. Additional lanes at the intersection or on the ramp may be warranted.

5. Turn Lanes – Exclusive left- and/or right-turn lanes may often be included on the crossroad and possibly on the ramp itself. Where additional turn lanes may be warranted in the future, consideration should be given to providing sufficient right of way to allow for the construction of future turn lanes. See Section 6-3.0 for more information.

6. Signalization – Where queuing at one intersection is long enough to affect operations at another, consideration should be given to providing a larger separation, interconnected signals, or a four-phase overlap signal design.

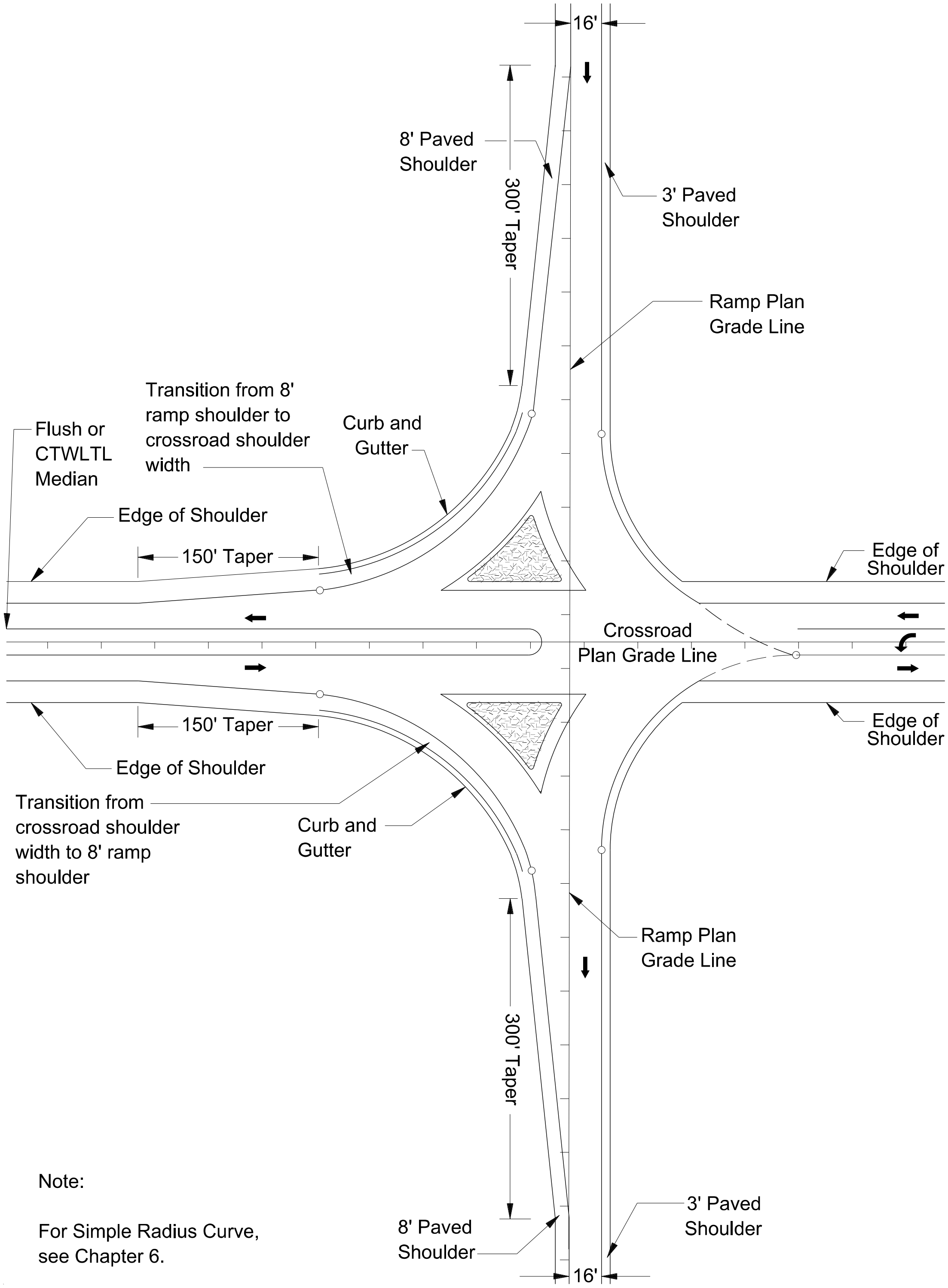
7. Design Vehicle – All radius returns and left-turn control radii for ramp terminals should typically be designed using a WB-67 design vehicle. However, adequate radii for vehicle operation should be balanced against the needs of pedestrians and the presence of right of way and/or environmental constraints.

8. Typical Designs – Figures 7-3-D and 7-3-E illustrate typical ramp terminals for a diamond interchange. Figure 7-3-D illustrates a 3-lane crossroad, and Figure 7-3-E illustrates a 5-lane crossroad with a Continuous Two-way Left-Turn Lane (CTWLTL).

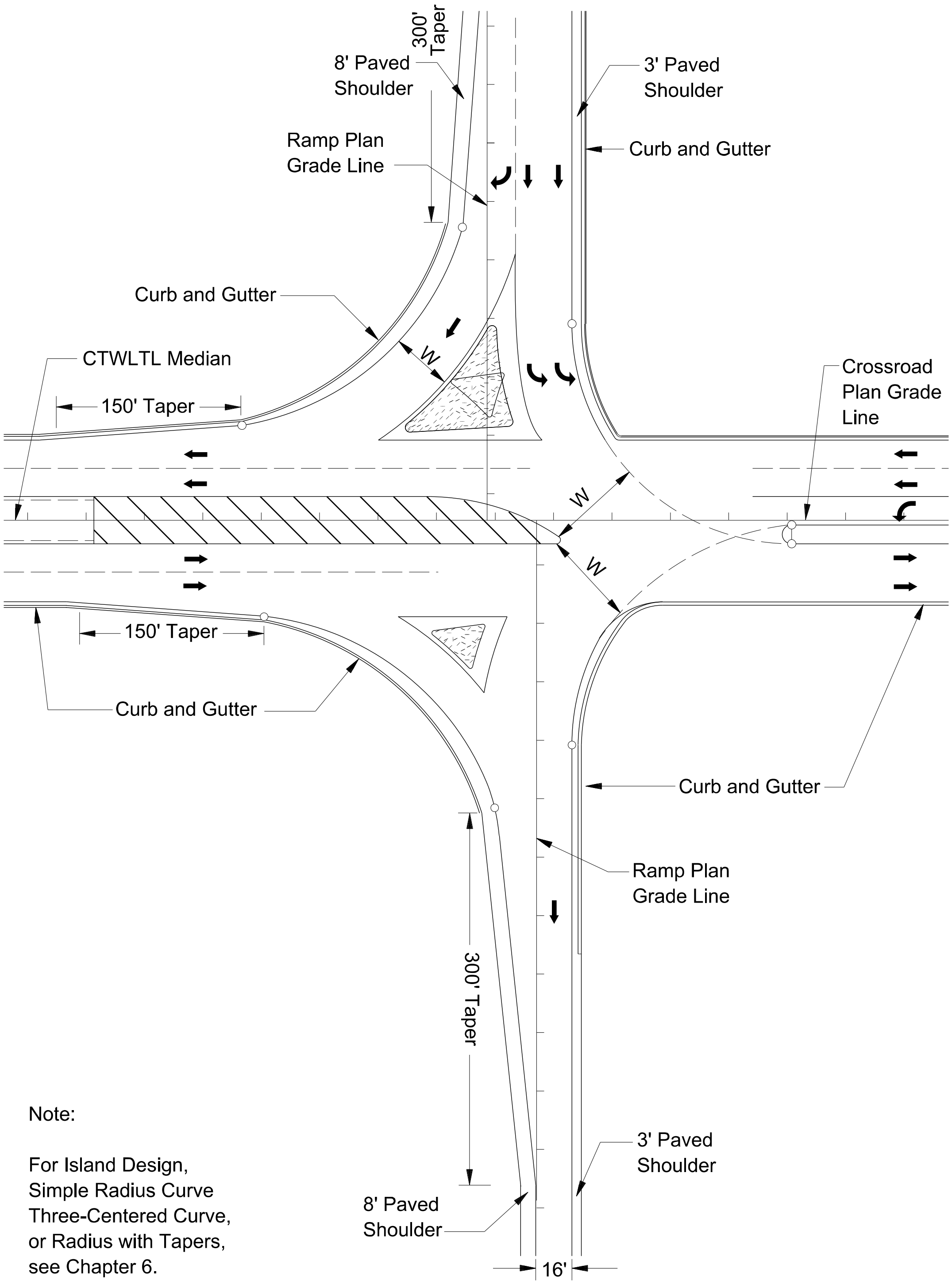
9. Wrong-Way Movements – Wrong-way movements may originate at the ramp terminal onto an exit ramp. To minimize their probability, the intersection should be designed to discourage this movement, and the exit ramp should be signed according to the criteria in the *MUTCD*.

10. Access Control – Proper access control should be provided along the crossroad near the ramp terminal in order to ensure that the intersection has approximately the same degree of freedom and absence of conflict as the mainline itself. Figure 7-3-F illustrates the typical location of the no-access right of way line at ramp terminals. See the *Access Management Manual* for minimum criteria.

    Many interchanges were initially constructed in Mississippi when the surrounding area was rural in character. Since that time, the area may have become suburban or urban. As indicated in the *Access Management Manual*, the Department has adopted different criteria for the access control at urban and rural interchanges. However, it is Department policy that a change in area character alone is not a sufficient justification to alter the location of a no-access right of way line along the crossroad.

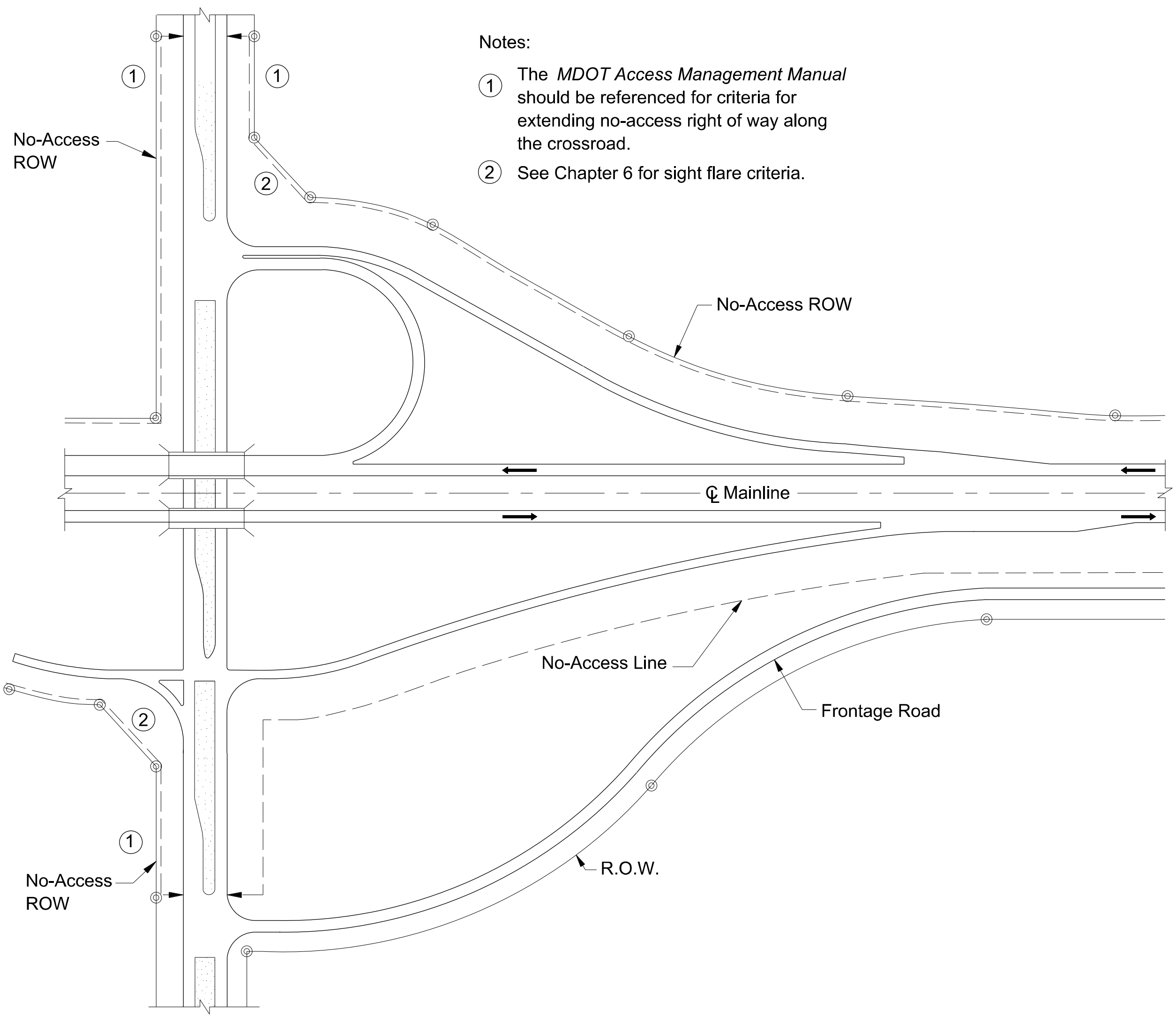


RAMP TERMINAL—DIAMOND INTERCHANGE
(3-Lane Crossroad)
Figure 7-3-D



Note: See Chapter 6, "At-Grade Intersections", for additional geometric details.

RAMP TERMINAL INTERSECTION — DIAMOND INTERCHANGE
(5-Lane TWLTL Crossroad — Signalized Intersections)
Figure 7-3-E



NO-ACCESS RIGHT OF WAY AT INTERCHANGE
Figure 7-3-F

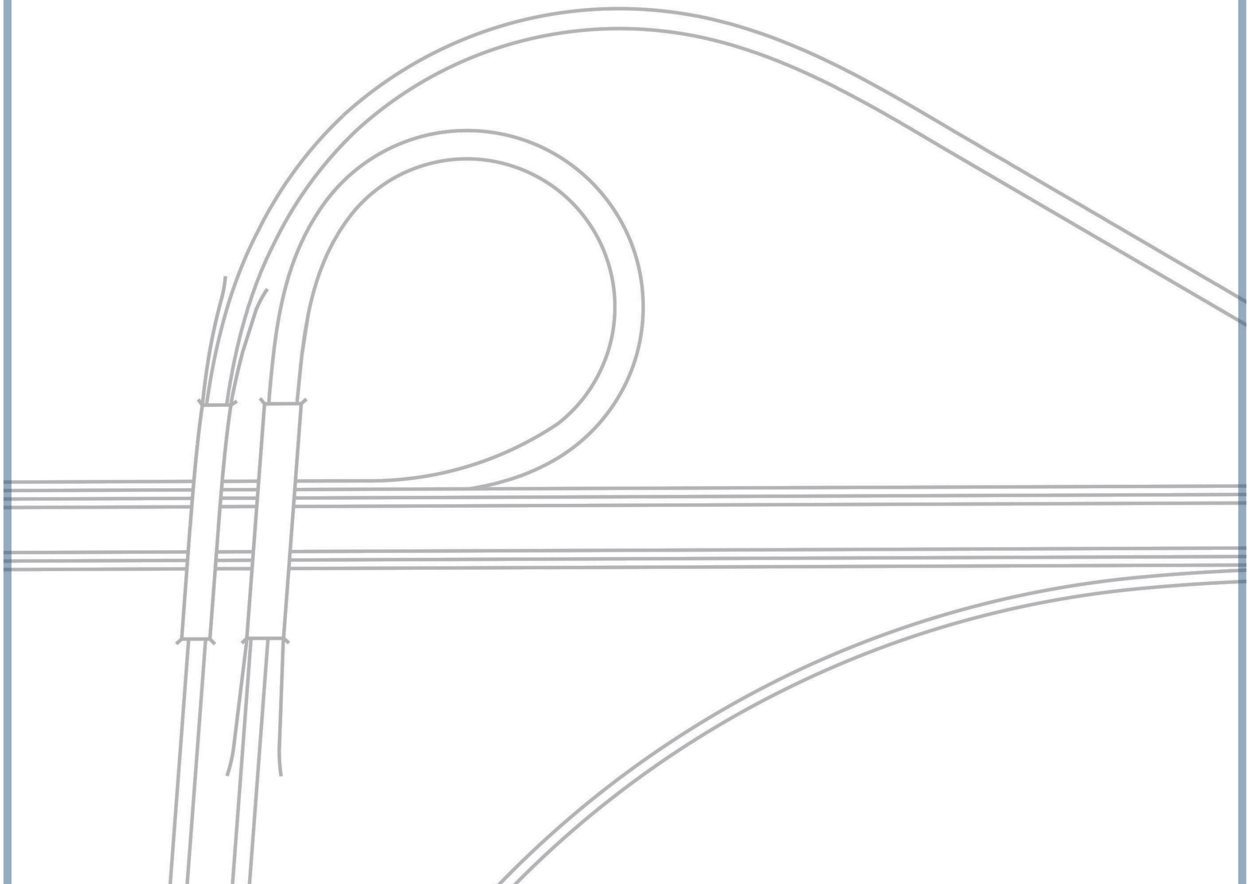
7-4.0 NEW/REVISED ACCESS POINTS ON THE INTERSTATE SYSTEM

Each entrance and exit point on the mainline of an interstate facility is defined as an access point (e.g., diamond interchanges have four access points). A change in the interchange configuration is considered a revised access, even though the number of access points may not change (e.g., replacing a loop ramp with a semi-directional ramp).

The Department's goal is to maintain an acceptable LOS, safety, and mobility on its interstate system, which is accomplished by controlling access onto the system. All proposed new or revised access points to the interstate system will require an Interstate Access Request (IAR). The IAR is completed by the Planning Division and formally approved by FHWA. Procedures for the development and the content of an IAR for new or revised access points to the interstate system are outlined within the FHWA policy memo and policy attachment (both dated May 22, 2017). Although not updated, use of guidance within the 2010 FHWA *Interstate System Access Informational Guide* can be beneficial toward the development and approval of an IAR. Ultimately, the interchange design and related access points as outlined with the approved NEPA & IAR documents are to be carried forward into design plan development.

7-5.0 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2018.
2. *Access Management Manual*, Mississippi Department of Transportation, 2010.
3. *Interstate System Access Informational Guide*, Federal Highway Administration, 2010.
4. *Freeway and Interchange: Geometric Design Handbook*, Institute of Transportation Engineers, Washington DC, 2005.
5. *Highway Capacity Manual 2010*, Transportation Research Board, 2010.
6. *Manual on Uniform Traffic Control Devices*, National Advisory Committee on Uniform Traffic Control Devices, 2009.
7. *FHWA-HRT-09-060 Alternative Intersections/Interchanges Informational Report,* FHWA, 2017.
8. *Freeway and Interchange Geometric Design Handbook,* ITE, 2006.



CHAPTER 8

Special Design Elements

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Chapter 8
SPECIAL DESIGN ELEMENTS

This chapter discusses the design criteria and procedures for erosion control, rumble strips/stripes, weigh stations, rest areas, airway/roadway clearances, and bicycle accommodations. Proper design of these elements should contribute to roadway safety and improve the operational efficiency of the facility.

8-1.0 ROADSIDE DEVELOPMENT

8-1.01 Erosion Control

Most roadway projects propose that certain erosion control measures be implemented (e.g., providing vegetation cover on disturbed soil areas), which should help prevent soil erosion, minimize sediment, and improve the appearance of the roadside.

8-1.01.01 Construction Plan Sheets

For all projects with a minimum of one acre of disturbed area, the following information should be used in assembling the Contract Plans:

1. Summary of Quantities Sheets – Estimated quantities for erosion control items should be shown on the Summary of Quantities sheets. The pay items and quantities in the plans are provided strictly as a toolkit for the contractors' bidding purposes. The actual pay items and quantities used should be determined by the successful contractor.

2. Plan-Profile Sheets – Permanent erosion control items (e.g., ditch treatments, permanent riprap) and temporary erosion control items (e.g., super silt fence, silt basins) for sensitive areas, such as bridge sites, should be shown on the plan-profile sheets. Only the quantities for permanent erosion control items should be included on the plan-profile sheets.

3. Temporary Erosion Control Plan Sheets – Temporary erosion control plan (ECP) sheets should be prepared for most projects with a disturbed area equal to or greater than one acre. These plans may be omitted for minor projects with no ground-disturbing activities; however, the Field Inspection Team should evaluate specific erosion control needs and determine what erosion control measures and sheets should be included in the plans. ECP sheets (without contours) should be submitted with the Storm Water Permit Application to MDEQ for all projects with a disturbed area equal to or greater than five acres.

   The temporary ECP sheets should be developed from the plan-profile sheets, showing only right of way lines and right of way markers, alignment, edge of pavement, construction limits, proposed contours and existing contours beyond the construction limits, drainage structures, silt basins, streams, lakes, wetlands, and vertical alignments with grade percentages. For projects that have no elevation change, such as those in the Delta, contours may not be necessary. On the ECP sheets, MicroStation levels with other information from the plan-profile sheets, such as labels and text in the plan view and profile notes, should be turned off. Working sheet numbers should begin with ECP-3 to correspond with worksheet 3 of the plan-profile sheets. Similarly, the Bridge Division should provide ECP-BR sheets that correspond to the bridge plans and roadway ECP sheets for the proposed bridge locations. An impaired waters stamp should be shown on the corresponding ECP and plan-profile sheets for any stream that is on the 303(d) list, or any stream where a Total Maximum Daily Load (TMDL) has been established. Other items shown on the ECP sheets may include, but are not limited to:

a. riprap at bridge sites, special ditches, streams, and slopes
b. grassed earth berms for clean water diversions
c. silt fence/super silt fence at bridge sites
d. silt basins
e. drainage areas

An erosion control recap block should be provided on each ECP sheet and should remain blank for the contractor's use. For all locations where specific details are not included in the temporary erosion control plans, the contractor should decide what measures are to be taken after the contract has been awarded.

4. <u>Vegetation Schedule Sheet</u> – The designer should include in the plans the appropriate vegetation schedule sheet, which specifies work items to be performed, pay item numbers, rates of application, seasonal requirements, and any special notes related to the project. The vegetation schedule may include the following items:

a. topsoil application
b. ground preparation
c. fertilizing
d. seeding
e. mulching
f. solid sodding
g. watering
h. insect pest control

A typical vegetation schedule is illustrated in Figure 8-1-A. The appropriate vegetation schedule should be chosen for the project type and district and included with the *Special Design* sheets.

5. <u>*Standard Drawing*s</u> – Typically, all erosion control detail standard drawings should be included in the Contract Plans. However, the Field Inspection Team should determine if the use of Standard Drawing ECD-17, "Temporary Culvert Stream Crossing", will be allowed before inserting this sheet into the final plan set.

PRELIMINARY NOT FOR CONSTRUCTION

VEGETATION SCHEDULE

	EROSION CONTROL ITEMS		SEASONAL APPLICATIONS-DATES & RATES				REQUIREMENTS
			SPRING & SUMMER		FALL & WINTER		
	PAY ITEM NO.	ITEMS	RATES	DATES	RATES	DATES	
	212-B001	STANDARD GROUND PREPARATION	PER SQ.YD.	MARCH 1 TO SEPTEMBER 1	PER SQ.YD.	SEPTEMBER 1 TO MARCH 1	GROUND PREPARATION REQUIRED ON AREAS TO RECEIVE SOLID SODDING OR SEEDING, AS APPLICABLE.
	225-B001	AGRICULTURAL LIMESTONE	TONS/ACRE	MARCH 1 TO SEPTEMBER 1	TONS/ACRE	SEPTEMBER 1 TO MARCH 1	LIMESTONE SHALL BE MECHANICALLY SPREAD UNIFORMLY AND INCORPORATED INTO THE SOIL PRIOR TO PLANTING.
	225-A001	COMBINATION FERTILIZER (13-13-13)	250 LBS./ACRE	MARCH 1 TO SEPTEMBER 1		SEPTEMBER 1 TO MARCH 1	FERTILIZER SHALL BE MECHANICALLY SPREAD UNIFORMLY AND INCORPORATED INTO THE SOIL PRIOR TO PLANTING.
(1)	213-C001	SUPERPHOSPHATE	0.5 TONS/ACRE (EST.)	MARCH 1 TO DECEMBER 1			SUPERPHOSPHATE (FOR BID ITEM PURPOSES).
(2)	225-A001	SEEDING (BERMUDAGRASS)	_ LBS./ACRE	MARCH 1 TO SEPTEMBER 1	_ LBS./ACRE		SEED REQUIRED ON DISTURBED AREAS. UNHULLED SEED MAY BE REQUIRED DURING THE DORMANT SEASON AS DIRECTED.
	225-A001	SEEDING (TALL FESCUE)	_ LBS./ACRE	MARCH 1 TO SEPTEMBER 1	_ LBS./ACRE		SEED REQUIRED ON DISTURBED AREAS.
(3)	225-A001	SEEDING (BAHIAGRASS)	_ LBS./ACRE	MARCH 1 TO SEPTEMBER 1	_ LBS./ACRE		SEED REQUIRED ON DISTURBED AREAS. UNHULLED SEED MAY BE REQUIRED DURING THE DORMANT SEASON AS DIRECTED.
(3)	225-A001	SEEDING (CRIMSON CLOVER)	_ LBS./ACRE	MARCH 1 TO SEPTEMBER 1	_ LBS./ACRE	SEPTEMBER 1 TO MARCH 1	SEED REQUIRED ON DISTURBED AREAS. UNHULLED SEED MAY BE REQUIRED DURING THE DORMANT SEASON AS DIRECTED.
	225-C001	MULCH - VEGETATIVE MULCH	2 TONS/ACRE (EST.)	MARCH 1 TO SEPTEMBER 1	2 TONS/ACRE (EST.)	SEPTEMBER 1 TO MARCH 1	THE ENGINEER WILL DESIGNATE THE RATES OF APPLICATION (SEE SUBSECTION 215.03.3).
	216-A001	SOLID SODDING	PER SQ.YD.	MARCH 1 TO SEPTEMBER 1	PER SQ. YD.	SEPTEMBER 1 TO MARCH 1	SOLID SOD REQUIRED ON AREAS SPECIFIED IN THE CONTRACT OR BY THE ENGINEER.
	219-A001	WATERING	20 GALS./S.Y. (EST.)	MARCH 1 TO SEPTEMBER 1	20 GALS. S.Y. (EST.)	SEPTEMBER 1 TO MARCH 1	TO BE USED AS DIRECTED ON THE PLANTING AND ESTABLISHING SOLID SOD.
(5)	220-A001	INSECT PEST CONTROL	PER ACRE		PER ACRE		SEE SECTION 220.
		TEMPORARY EROSION CONTROL ITEMS					
	226-A001	LIGHT GROUND PREPARATION	PER SQ.YD.		PER SQ.YD.		APPROXIMATELY HALF SQ. YD. STANDARD GROUND PREPARATION
	226-A001	COMBINATION FERTILIZER (13-13-13)					QUANTITY BASED ON LIGHT GROUND PREPARATION
	226-A001	SEEDING (BROWN TOP MILLET)			———	———	QUANTITY BASED ON LIGHT GROUND PREPARATION
	226-A001	SEEDING (RYE GRASS)	———	———	25 LBS./ACRE	SEPTEMBER 1 TO MARCH 31	QUANTITY BASED ON LIGHT GROUND PREPARATION
	226-A001	SEEDING (OATS)	———	———	90 LBS./ACRE	SEPTEMBER 1 TO DECEMBER 15	QUANTITY BASED ON LIGHT GROUND PREPARATION
	226-A001	VEGETATIVE MATERIAL FOR MULCH	2 TON /ACRE (EST.)		2 TON /ACRE (EST.)		QUANTITY BASED ON LIGHT GROUND PREPARATION

(1) ALL AREAS THAT HAVE BEEN VEGETATED, UNDER THIS CONTRACT FOR AT LEAST (60) SIXTY DAYS, SHALL RECEIVE ADDITIONAL APPLICATION(S) OF FERTILIZER(S) OF THE TYPE(S) AND RATE(S) OF APPLICATIONS AS DETERMINED BY SOIL TESTS OR AS DIRECTED DURING THE GROWING SEASONS THE CONTRACT IS IN FORCE. GROUND PREPARATION WILL NOT BE REQUIRED FOR THE ADDITIONAL APPLICATIONS. PAYMENT FOR ALL FERTILIZERS ACCEPTABLY APPLIED AS AN ADDITIONAL APPLICATION(S) WILL BE MADE IN ACCORDANCE WITH SUPERPHOSPHATE BID ITEM 213-C001.

(2) PROPOSAL QUANTITIES ESTIMATED ON THE BASIS THAT 100% OF THE ACREAGE WILL BE SEEDED.

(3) PROPOSAL QUANTITIES ESTIMATED ON THE BASIS THAT 50% OF THE ACREAGE WILL BE SEEDED.

(4) PROPOSAL QUANTITIES ESTIMATED ON THE BASIS THAT OF THE ACREAGE WILL BE SEEDED.

(5) QUANTITY ESTIMATED ON THE BASIS THAT 50% OF THE ACREAGE VEGETATED MAY REQUIRE TREATMENT.

(6) SOW, IN ADDITION TO OTHER SPECIFIED SEEDS, ON HIGH FILL AND BACKSLOPE AREAS SELECTED BY ENGINEERS DURING CONSTRUCTION. PROPOSAL QUANTITIES ESTIMATED ON THE BASIS THAT 50% OF THE ACREAGE WILL BE SEEDED.

(7) THE ACTUAL RATE/ACRE TO BE DETERMINED BY SOIL TEST DURING CONSTRUCTION.

(8) BERMUDA GRASS SEED MIX - 50% HOLLED, 50% UNHOLLED.

(9) BAHIAGRASS WILL NOT BE PERMITTED AS A MULCH MATERIAL.

(10) PROPOSAL QUANTITIES ESTIMATED ON THE BASIS THAT 75% OF THE ACREAGE SEEDED MAY REQUIRE TOPSOIL.

(11) PROPOSAL QUANTITIES ESTIMATED ON THE BASIS THAT 50% OF THE ACREAGE SEEDED MAY REQUIRE TOPSOIL.

(12) THIS ITEM TO BE OMITTED ON AREAS WITHIN 30' FROM EDGE OF PAVEMENT.

(13) THIS ITEM TO BE OMITTED ON AREAS SELECTED BY THE ENGINEER.

MDOT MISSISSIPPI DEPARTMENT OF TRANSPORTATION

DESIGNED BY: N/A
DETAILED BY: N/A
CHECKED BY: N/A
DATE: N/A

FMS CON: N/A/N/A
PROJECT NO.: N/A
COUNTY: N/A

VEGETATION SCHEDULE

WK. NO. VS-1
SHEET NO. 45

PLAN SHEET

2/7/2020 12:51 PM MDOT Vegetation Sheets.dgn

EXAMPLE VEGETATION SCHEDULE
Figure 8-1-A

8-1.01.02 Estimating Quantities

Typical procedures for estimating erosion control items are as follows:

1. Topsoil – The total quantity of topsoil (cubic yards) should be determined by multiplying each area (square feet) by the recommended topsoil thickness (feet) and dividing the total by 27 to convert to cubic yards. The cumulative sum of these calculations should yield the total quantity of topsoil for the project. This value should be multiplied by a factor of 1.25 to account for 25% shrinkage in material.

2. Grassing – The project's total disturbed area should be estimated in acres, including all permanent roads and temporary detour roads. A temporary grassing pay item should also be included for this same area.

3. Superphosphate – The quantity of superphosphate (tons) should be calculated by first obtaining the area for grassing in acres, then multiplying the area by the rate of application (tons per acre).

4. Agricultural Limestone – The quantity of agricultural limestone (tons) should be calculated by first obtaining the area for grassing in acres, then multiplying the area by the rate of application (tons per acre).

5. Vegetative Materials for Mulch – The quantity of vegetative materials for mulch (tons) should be calculated by first obtaining the area for grassing in acres, then multiplying the area by the rate of application (tons per acre).

6. Solid Sodding – The quantity of solid sodding should be measured by the square yard for areas where it is specifically shown in the plans (e.g., adjacent to concrete paved ditches, paved flumes, paved aprons at inlets, as a proposed ditch treatment).

7. Watering – The quantity of watering (gallons) should be determined by first multiplying the area of solid sodding (square yards) by the rate of application (gallons per square yards) of water, then rounding to the nearest whole thousand gallons.

8. Insect Pest Control – The quantity of insect-pest-control area (acres) should be calculated by first obtaining the grassing area in acres, then dividing the total acreage by two.

9. Temporary Ditch Checks – Quantities for the following temporary ditch check items should typically be estimated:

   a. Temporary Erosion Checks
   b. Wattles (20 inches)
   c. Triangular Silt Dike
   d. Sandbags
   e. Riprap for Erosion Control

   The plan quantity for each of these items should be based on 1/5 of the total number of estimated ditch checks.

10. Temporary Silt Fence – A sufficient plan quantity of temporary silt fence should be estimated for the following:

    a. all fill sections within the project limits
    b. sensitive areas (e.g. bridges discussed in Section 8-1.01.01)
    c. ditch checks as shown in *Standard Drawings*

    The estimated plan quantity for temporary silt fence typically should be determined by adding the lengths from the first two items above, increasing this amount by 10%, and rounding the final quantity to the nearest 50 feet. The additional 10% is intended to account for replacement of damages, relocation due to construction phasing, and ditch checks.

11. Inlet Protection – The following inlet protection items should be included:

    a. Inlet Siltation Guard
    b. Temporary Erosion Checks
    c. Wattles (12 inches)
    d. Sandbags

    The estimated plan quantity for each of these items should be based on 1/4 of the total number of inlets. Additional quantities of silt fence, wattles, and sand bags may be provided for inlet protection during early construction phases as shown in the ECP detail sheets.

12. Other Erosion Control Pay Items – Examples of other recommended temporary erosion control pay items may include, but are not limited to:

    a. Solid Sodding*
    b. Erosion Control Blanket, Type 1*
    c. Super Silt Fence*
    d. Turbidity Barrier
    e. Silt Basin, Type D
    f. Coir Fiber Baffle
    g. Sediment Retention Barrier
    h. Temporary Stream Diversion
    i. Remove and Reset Riprap
    j. Loose Riprap, Size 300
    k. Sediment Control Stone

    * Due to their cost, these items should typically be used only when recommended by the Office Review Team.

8-1.02 Special Side Ditch Treatment

Certain physical conditions (e.g., steep ditch gradients, large volumes of surface runoff) make it difficult to establish vegetation before erosion occurs. Under these conditions, special side ditch treatments may be recommended. An analysis should be conducted and the appropriate side ditch treatment designed in accordance with the criteria and procedures found in the following sections. Where unusual conditions exist, the Hydraulics Branch should be consulted for guidance in the design of ditches, channels, and other erosion control measures.

8-1.02.01 Design Criteria

The main factors affecting ditch design include:

1. ditch gradient
2. surface runoff (drainage area)
3. ditch side slopes
4. type of soil

The alternative side ditch treatments for erosion control are as follows:

1. normal seeding and mulching
2. ditch liners
3. solid sod
4. soil reinforcing mats
5. riprap
6. paved ditches
7. erosion control blanket

8-1.02.02 Drainage Areas

The drainage area should include any portion of the pavement surface that is sloped toward the ditch, all construction slopes, and any area beyond the construction limits that drains toward the ditch. Table 8-1-A provides criteria for side ditch treatments based on the computed drainage area.

8-1.02.03 Ditch Treatment Design

The design of side ditch treatment typically involves the following steps:

1. The proper columns in Table 8-1-A should be determined.

2. The appropriate ditch gradient (percent) in Table 8-1-A should be selected.

3. The upper limit (area) for each type of treatment should be identified.

4. The area determination process should be started at the upstream end of the drainage area and continued until the area reaches the upper limit for normal seeding and mulching (no special treatment). The ditch liner treatment should typically begin at this location.

5. The area determination method should be continued until the upper limit for ditch liner treatment is reached. The solid sod should typically begin at this point.

6. The area determination method should be continued until the upper limit for solid sod is reached. Where soils permit, this upper limit can be increased by 25%. The soil reinforcing mat treatment should be used until this new limit is reached. Otherwise, either riprap or paved ditch treatment should be provided and continued to the point of termination.

7. Where the ditch reaches an outlet, the indicated ditch treatment should be ended and the area determination method started again from a zero reading at the upstream end of the next drainage area. Ditch treatments are generally run to the nearest 25-foot interval.

Details of typical ditch treatments are provided in the *Standard Drawings*. Where moderate capacity is anticipated, a simple V-shape ditch should be used. For situations involving relatively large volumes of surface runoff, a flat-bottom ditch with the appropriate treatment should be considered.

The location and type of special ditch treatments should be noted on the plan sheets. Table 8-1-B provides the plan sheet symbols and the units of measurement for each type of side ditch treatment. If plan sheet information (e.g., the topography, construction notes, survey data) causes the sheet to become difficult to read, the ditch treatment symbols may be omitted and an itemized list of quantities should be prepared that identifies each type of treatment by stationing.

8-1.03 Silt Basins

Where feasible, clean water diversions should be used to reduce the amount of off-site water flowing through silt basins on the site, thereby reducing the size of the proposed silt basins. Clean water diversion may also eliminate the need for a silt basin at some locations. Figure 8-1-B illustrates typical clean water diversions for cut and fill sections.

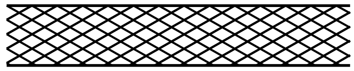
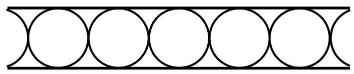
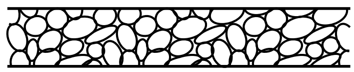
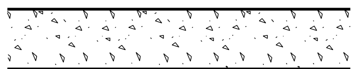
Table 8-1-A
CRITERIA FOR SIDE DITCH TREATMENT

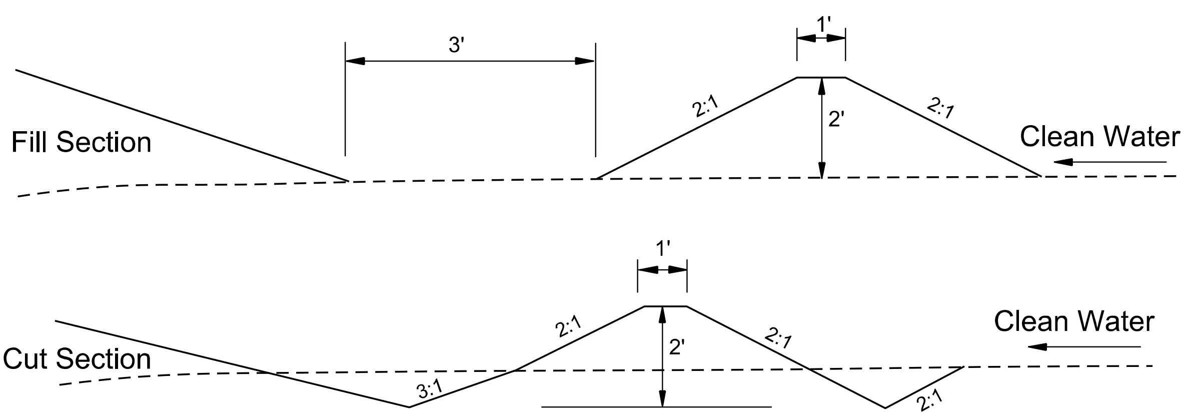
Ditch Grade (%) ⑥	UPPER AREA LIMIT FOR SIDE DITCH TREATMENT (acres)①② — Foreslope — Backslope 6:1 — 6:1, 10:1 — 3:1: No Treatment	Ditch Liner	Soil Reinforcing ③ Mat ④⑤	Foreslope — Backslope 6:1 — 4:1, 6:1 — 3:1: No Treatment	Ditch Liner	Soil Reinforcing ③ Mat ④⑤	Foreslope — Backslope 4:1 — 4:1: No Treatment	Ditch Liner	Soil Reinforcing ③ Mat ④⑤	Foreslope — Backslope 4:1 — 3:1: No Treatment	Ditch Liner	Soil Reinforcing ③ Mat ④⑤	Ditch Grade (%) ⑥
0.2	1.73	3.14	7.39	1.43	2.62	6.15	1.14	2.08	4.87	0.99	1.83	4.22	0.2
0.3	1.73	3.56	9.07	1.46	2.97	7.51	1.16	2.37	5.98	1.01	2.05	5.19	0.3
0.4	1.83	3.98	10.45	1.51	3.24	8.65	1.21	2.62	6.87	1.04	2.27	5.98	0.4
0.5	1.75	4.00	10.67	1.46	3.31	8.87	1.16	2.64	7.02	1.01	2.30	6.10	0.5
0.6	1.53	3.51	9.49	1.26	2.92	7.86	1.01	2.32	6.25	0.89	2.03	5.46	0.6
0.7	1.33	3.14	8.55	1.09	2.57	7.02	0.89	2.08	5.61	0.77	1.80	4.87	0.7
0.8	1.24	2.87	7.81	1.04	2.40	6.45	0.82	1.90	5.14	0.72	1.66	4.45	0.8
0.9	1.14	2.64	7.19	0.94	2.20	5.98	0.74	1.73	4.74	0.67	1.53	4.13	0.9
1.0	1.04	2.45	6.72	0.84	2.03	5.56	0.69	1.63	4.42	0.59	1.41	3.85	1.0
1.5	0.77	1.88	5.19	0.62	1.53	4.30	0.49	1.24	3.41	0.44	1.09	2.97	1.5
2.0	0.62	1.56	4.37	0.52	1.28	3.61	0.40	1.01	2.89	0.37	0.91	2.50	2.0
2.5	0.52	1.36	3.83	0.42	1.11	3.16	0.35	0.89	2.52	0.30	0.77	2.20	2.5
3.0	0.44	1.16	3.36	0.40	0.99	2.79	0.30	0.79	2.22	0.27	0.69	1.93	3.0
3.5	0.42	1.09	3.09	0.35	0.91	2.57	0.27	0.72	2.03	0.25	0.62	1.78	3.5
4.0	0.37	0.96	2.72	0.30	0.79	2.25	0.25	0.64	1.79	0.22	0.54	1.56	4.0
4.5	0.35	0.89	2.52	0.27	0.74	2.10	0.22	0.59	1.66	0.20	0.49	1.46	4.5
5.0	0.30	0.82	2.35	0.27	0.69	1.98	0.20	0.54	1.58	0.17	0.47	1.36	5.0
5.5		0.57	2.25		0.47	1.85		0.37	1.48		0.32	1.28	5.5
6.0		0.49	2.03		0.42	1.71		0.32	1.33		0.30	1.16	6.0
7.0		0.44	1.80		0.37	1.48		0.30	1.21		0.27	1.04	7.0
8.0		0.37	1.51		0.32	1.26		0.25	0.99		0.22	0.86	8.0
9.0		0.32	1.26		0.27	1.06		0.20	0.84		0.20	0.74	9.0
10.0		0.25	1.01		0.22	0.84		0.17	0.67		0.15	0.59	10.0

Notes:

① *Values in this table are the upper limits for each particular type of side ditch treatment.*
② *Values in this table are for average erosion resistant soils. For highly-erosive soils, only 60% of the upper area limit should be used.*
③ *For solid sod, use 80% of the upper area limit.*
④ *Riprap or paving should begin beyond this limit.*
⑤ *FHWA Hydraulic Toolbox can be used to assist in this calculation.*
⑥ *Riprap or paving should begin where abrupt changes in ditch grades occur (e.g., 2.0%± to 6.0%±).*

Table 8-1-B
SYMBOLS AND UNITS OF MEASUREMENT FOR SIDE DITCH TREATMENTS

Ditch Treatment	Plan Sheet Symbol	Unit of Measurement
Ditch Liner		square yard
Solid Sod		square yard
Soil Reinforcing Mat		square yard
Riprap		ton
Paved Ditch (PC concrete)		cubic yard
Geotextile Fabric		square yard



CLEAN WATER DIVERSIONS
Figure 8-1-B

8-1.04 Clearing Limits Adjacent to Streams

The Department’s Storm Water Management Plan (SWMP) contains requirements to protect and preserve riparian buffers and native vegetation adjacent to streams. A riparian buffer is a vegetated area near a stream, usually forested, that helps protect the stream from the impact of adjacent land uses. Riparian buffers intercept sediment, nutrients, pesticides, metals, and other materials in surface water runoff. Riparian buffers also provide wildlife habitat and corridors in primarily agricultural areas and reduce erosion by providing bank stabilization.

To protect these sensitive areas, clearing adjacent to streams (which are identified by blue solid or dashed lines on USGS quad maps) should be limited, where feasible, to no closer than 50 feet to the top of the banks and no more than 10 feet from the construction limits. For those projects where it is not feasible to achieve the desirable clearing limits, the Roadway Design Environmental Engineer should assess these sensitive areas on a case-by-case basis to determine if additional erosion and sediment controls may be warranted.

The Roadway Design Environmental Engineer should identify the streams along which riparian buffers are to be protected. For any such streams, the ECP sheets discussed in Section 8-1.01.01 should include additional plan sheets produced at a larger scale to clearly indicate the clearing limits.

The Roadway Design Environmental Engineer should also identify which streams are on the Mississippi Department of Environmental Quality (MDEQ) Section 303(d) List of Impaired Water Bodies and streams where Total Maximum Daily Loads (TMDLs) have been determined, then recommend best management practices (BMPs) to lessen the roadway construction impacts on the impaired streams. Section 303(d) of the *Clean Water Act* requires Mississippi to identify those waters within its jurisdiction that are impaired for any number of reasons, which may or may not pertain to Department operations. The Roadway Design Environmental Engineer should also provide the Environmental Division the drawings necessary to obtain wetland permits, update permit information in the Department's on-line project management tool (PDPMv2), and initial the permit block on title sheets for roadway projects. Project surveys and wetland information should be provided to the Roadway Design Environmental Engineer as soon as they become available.

8-1.05 Other Erosion Control

8-1.05.01 Riprap

To prevent excessive erosion, riprap may be proposed at various locations within the roadway right of way. The Hydraulics Branch should provide guidance in the use of riprap at the locations listed below. The District, Construction Division, Environmental Division, and FHWA may also provide input on these locations:

1. culvert outfall and intake ends
2. bends in stream channels
3. abrupt flowline changes
4. excessive flowline grades
5. the point of confluence of two or more streams or channels
6. existing erosion problem areas

Riprap and geotextile fabric at bridge sites (e.g., end bents, channel banks, guide banks) should be included in the bridge quantities and should not be included in the roadway quantities.

8-1.06 Landscaping

8-1.06.01 Responsibilities

Roadside development work may be specified during the project planning process. This work may include landscaping of the roadside or development of roadside rest areas. The Architectural Services Unit is responsible for the design of these projects and for the preparation of plans.

For landscaping work, plans typically include:

1. title sheet and general layout
2. summary of quantities
3. vegetation schedule
4. plan sheets showing individual plantings
5. applicable *Standard Drawings* (e.g., Typical Planting Details for Trees and Shrubs)

8-1.06.02 Safety Considerations

Adequate clearance should be provided beyond the edge of the traveled way for the recovery of out-of-control vehicles. This area is called the clear zone and should be free of any roadside obstacles. Therefore, roadside landscaping should not include any features that could be considered as obstacles or hazards. The application of the recommended clear zone criteria is presented in Section 9-2.0.

Trees with a diameter of four inches or larger, or those that have the potential to reach a diameter of four inches or larger during the life of the facility, should not be located within the recommended clear zone. In addition, the planting and maintenance of trees and shrubs should ensure that a driver's line of sight to signs and to decision points is not obscured and that the recommended sight distance is maintained.

8-2.0 RUMBLE STRIPS AND RUMBLE STRIPES

Rumble strips and rumble stripes are a proven, cost-efficient way to reduce crashes by alerting drivers of roadway departures with audible and vibratory warnings.

The following information is applicable to all construction and maintenance projects:

1. Rumble strips should be provided on the shoulders of new concrete pavement and open-graded friction courses.

2. Rumble stripes should be provided on any asphalt roadway with a minimum 2-foot paved shoulder width.

3. Rumble stripes or rumble strips may be omitted in the following cases:

    a. in residential or business areas

b. within the corporate limits of a city

c. where curb and gutter is present

d. in places where 2-foot paved shoulders cannot be provided

e. where lane widths are less than 11 feet

4. Where rumble strips or stripes are provided on bicycle routes, the following criteria should be addressed:

   a. Where bicycles share the road, a minimum 1-foot offset should be provided between the travel lane and the rumble strip.

   b. Where designated bicycle lanes are included, a minimum 4-foot lateral clearance should be provided from the rumble stripe to the outside edge of the paved shoulder.

   c. Where designated bicycle lanes are included, a minimum 5-foot lateral clearance should be provided from the rumble stripe to adjacent guardrail or other obstacles.

The *Standard Drawings* should be referenced for installation details of rumble strips and rumble stripes.

8-3.0 WEIGH STATIONS

8-3.01 Responsibilities

The need for a new weigh station facility should be evaluated whenever a proposed project involves an existing weigh station or crosses the state line. The Architectural Services Unit or assigned representative should contact the Department's Office of Enforcement for recommendations on a replacement facility.

If a new weigh station is warranted, the Architectural Services Unit should initiate and coordinate the necessary planning actions with the Law Enforcement Bureau, District, and other divisions, as needed. After site selection and completion of surveys, the Architectural Services Unit should be responsible for the design of the weigh station facility and preparation of the plans, specifications, and cost estimate. The site preparation for the weigh station should typically be included in the roadway contract. The building and facility construction should typically be let as a separate contract.

8-3.02 Design Responsibilities

The Architectural Services Unit should coordinate with the Roadway Design Division on the design of weigh station projects and the preparation of plans and specifications. Plans for weigh station projects typically include:

1. site preparation and drainage
2. buildings
3. scales

4. site lighting and electrical power
5. water and sewer installation
6. radio tower installation
7. signing and flagpole
8. erosion control and landscaping

8-3.03 Exit and Entrance Ramps

The exit and entrance ramp junctions for weigh stations should be modified versions of the typical ramp junction designs for interchange ramps. The exit ramp should be designed in accordance with the *Standard Drawings* using a single horizontal curve in advance of the dynamic scale. A minimum distance of 800 feet should be provided from the ramp gore to the dynamic scale to accommodate vehicular deceleration, which allows drivers to maintain a steady speed and achieve proper spacing between vehicles. The approach tangent preceding the dynamic scale should desirably be level and smooth for 200 feet to enhance scale performance. The ramp downstream from the dynamic scale should also be level for 100 feet.

An acceleration distance of approximately 2000 feet should be provided from the static scale to the gore nose of the highway to allow trucks to achieve an acceptable speed before entering the highway. The acceleration distance should be adjusted where the ramp junction is on a grade greater than 3% or where the highway Level of Service (LOS) is adversely affected by the truck's merging speed. Section 7-3.04 should be referenced for additional information.

8-4.0 REST AREAS

8-4.01 Responsibilities

If a new rest area is warranted or an existing rest area is rehabilitated, the Architectural Services Unit should initiate and coordinate the necessary planning actions with the District and other divisions, as necessary. After site selection and completion of surveys, the Architectural Services Unit should be responsible for the design of the rest area facility and preparation of the plans, specifications, and cost estimate.

For roadside rest areas, plans should typically include details on:

1. site preparation
2. buildings
3. water and sewer installations
4. other facilities

8-4.02 Roadway Design Elements

The following guidelines should also be considered for roadway design at rest areas:

1. Buffer Zone – To enhance patron safety and discourage people from stopping on the highway, the facility should be located a minimum distance of 30 feet away and desirably 150 feet from the highway shoulder to create a buffer zone to the nearest use area.

2. Internal Roadways – The internal rest area roadways should be designed with a desirable design speed of 25 miles per hour, but with no less than a design speed of 20 miles per hour.

3. Curbing – Curbing is recommended on internal roadways for delineation, to restrict illegal parking on the shoulder, and to minimize vehicular encroachment.

4. Accessibility for Disabled Individuals – See Section 14-2.06.05 for information on accessibility for disabled individuals.

5. Fencing – The rest area right of way should be fenced to prevent access to or from adjacent properties.

8-4.03 Entrance and Exit Ramp Junctions

Access to and from rest areas should be provided according to the typical ramp junction designs discussed in Section 7-3.04. The following additional information on entrance and exit ramp junctions should be considered:

- Exit Ramps – Intersection Sight Distance (ISD) should be provided for the exit maneuver. A minimum deceleration distance of 600 feet should be provided from the gore of the exit ramp to the car/truck divergence gore.

- Entrance Ramps – A minimum distance of 1200 feet should be provided from the truck parking area to the gore nose on the entrance ramp. This distance typically should allow a truck to accelerate to an acceptable speed before entering the highway. The acceleration distance should be adjusted where the ramp junction is on a grade greater than 3%, or where the highway LOS is adversely affected by the truck's merging speed.

Ramps should be designed to accommodate a WB-67 design vehicle. Ramp widths should typically be 16 feet wide; however, the ramp width criteria in AASHTO's *A Policy on Geometric Design of Highways and Streets* should be reviewed to determine the applicable ramp width for turning vehicles. Case II (which allows other vehicles to pass a stalled vehicle) and traffic condition "C" (a significant number of tractor-semitrailer combination trucks to govern the design) should be used.

8-4.04 Vehicle Parking

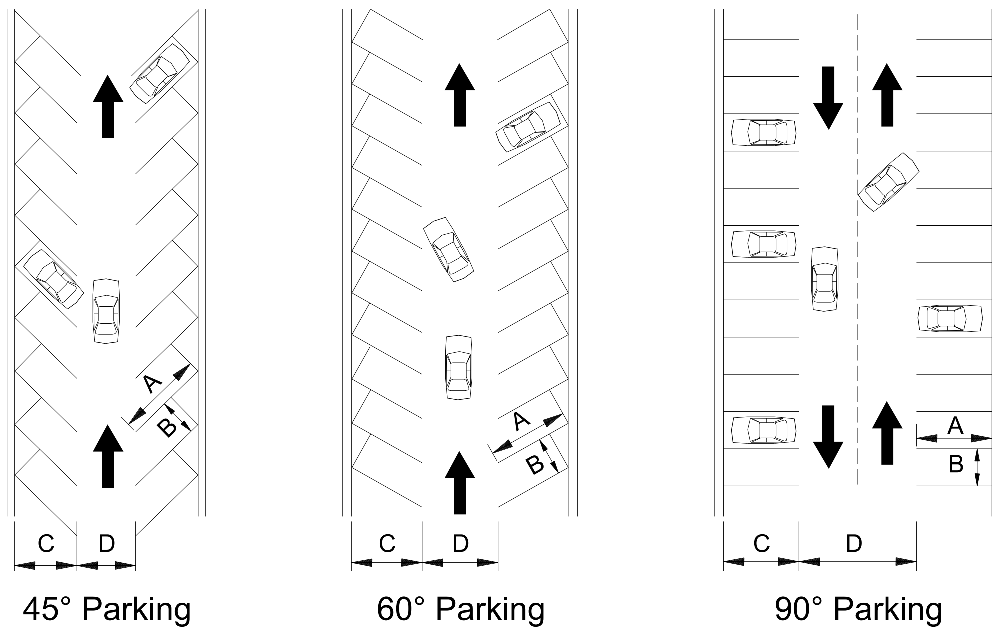
Parking lots should only be as large as recommended by design calculations while also providing a logical circulation pattern. Oversized lots can be confusing to drivers and can distract from the aesthetics of the rest area. Separate lots should be provided for trucks and cars. Typically, truck parking areas should be located to the rear of a site, except when the terrain or a scenic vista would be better served if passenger cars were located at the rear of a site.

Within the car parking area, a through roadway width of 20 feet is desired with diagonal parking generally on both sides of the roadway. Figure 8-4-A illustrates typical parking configurations for passenger cars. Standard curb and gutter should be provided within the vehicle parking area. Figure 8-4-B illustrates typical parking configurations for a WB-67 tractor-trailer truck.

8-4.05 Pavement/Shoulder Design

The concentration of heavy trucks braking on the ramps and inner roadways and the sharp turning maneuvers to enter parking stalls may result in special pavement designs. Where overflow truck parking is allowed on the ramp shoulders, the shoulder pavement should be designed for truck parking.

Standard curb and gutter should be used in passenger car parking areas and where curbs are desired to restrict trucks from parking on the shoulder. Island noses may be ramped for ease of maintenance.

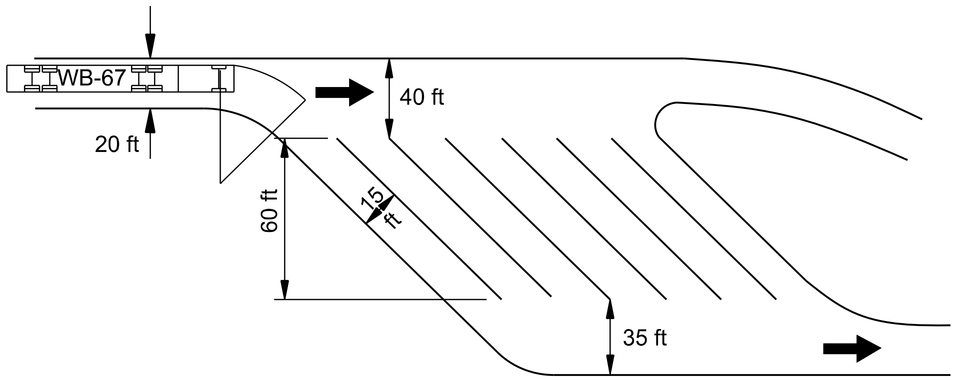


	45° Parking	60° Parking	90° Parking
A	25 ft	22 ft	18 ft
B	9 ft	9 ft	9 ft
C	17 ft	19 ft	18 ft
D	12 ft	15 ft	24 ft

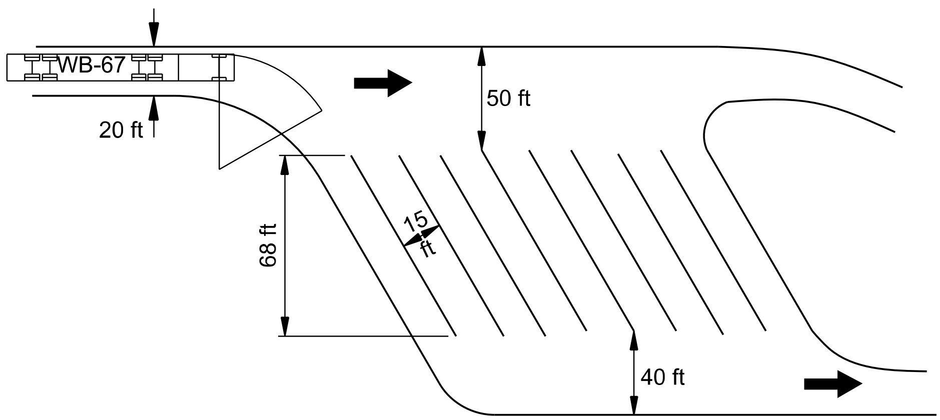
Notes:

1. *All dimensions shown are minimum.*
2. *Greater aisle widths may be desirable to accommodate high number of turning movements associated with short-term parking and through traffic.*

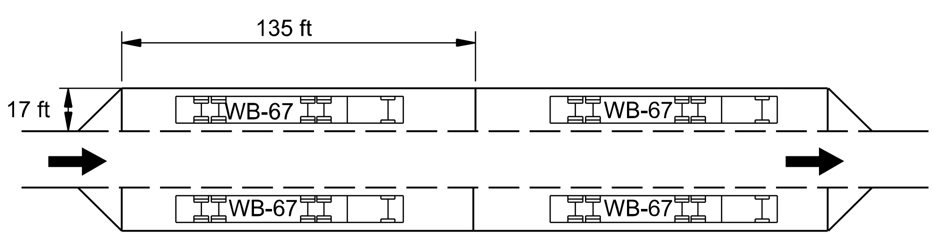
TYPICAL PASSENGER CAR PARKING
Figure 8-4-A



45° Truck Parking



60° Truck Parking



Truck Aisle Parking

Note: All dimensions shown are minimums.

TYPICAL TRUCK PARKING
Figure 8-4-B

8-5.0 AIRWAY/ROADWAY CLEARANCES

Roadways in the vicinity of airports should be designed to provide minimum clearance between the roadways and the navigable airspace. Navigable airspace is defined as the space above the approach surfaces adjacent to the runway. The bottom of the approach surface is established by a specified guide path ascending upward from the end of the runway. Roadway appurtenances (e.g., overhead signs, light standards) should not penetrate the navigable airspace. The following publications should be referenced for design criteria related to airway/roadway clearances:

- *Procedures for Handling Airspace Matters*, JO 7400.2J, Federal Aviation Administration, 2012
- *Safe, Efficient Use and Preservation of Navigable Airspace*, 14 CFR 77, Code of Federal Regulations

14 CFR 77.9 states that any person/organization who intends to sponsor any of the following construction or alterations must notify the Administrator of the FAA:

1. Any construction or alteration exceeding 200 feet above ground level:
   a. Within 20,000 feet of a public use or military airport that exceeds a 100:1 surface from any point on the runway of each airport with its longest runway more than 3200 feet
   b. Within 10,000 feet of a public use or military airport that exceeds a 50:1 surface from any point on the runway of each airport with its longest runway no more than 3200 feet
   c. Within 5000 feet of a public use heliport that exceeds a 25:1 surface
2. Any roadway, railroad, or other traverse way with a prescribed adjusted height that exceeds the above noted standards
3. When requested by the FAA
4. Any construction or alteration located on a public use airport or heliport, regardless of height or location

When a project meets one or more of the above conditions, the Department's Aeronautics Division should be contacted.

8-6.0 BICYCLE ACCOMMODATION

8-6.01 General

In rural areas, a dedicated bicycling space is typically not provided along roadways. In urban areas, bicycling space may be provided by a shared roadway with wide curb lanes or dedicated space (e.g., designated bicycle lanes). Separate bicycle facilities may be considered adjacent to high-volume roadways. Sidewalks should not be considered as bicycle facilities.

8-6.02 Bikeway Classifications

The following definitions apply to bikeway classifications:

1. Bicycle Lane or Bike Lane – A portion of a roadway that has been designated by striping and/or signing for exclusive use by bicyclists.

2. Bicycle Route – A roadway or bikeway designated by the jurisdiction having authority, either with a unique route designation or with bicycle route signs, along which bicycle guide signs may provide directional and distance information. Signs that provide directional, distance, and destination information for cyclists do not necessarily establish a bicycle route.

3. Shared Roadway – A roadway that is open to both bicycle and motor vehicle travel. A shared roadway may be an existing roadway, a street with wide curb lanes, or a roadway with paved shoulders.

4. Shared Use Path – A path physically separated from motorized vehicular traffic by an open space or barrier, and either within the roadway right of way or within an independent right of way. Shared use paths may be used by bicyclists, pedestrians, skaters, wheelchair users, or joggers, and are typically designed for 2-way travel.

5. Shared Lane – A lane of a traveled way that is open to bicycle travel and vehicular use.

6. Rail Trail – A shared-use path, either paved or unpaved, built within the right of way of a former railroad.

7. Rail with Trail – A shared-use path, either paved or unpaved, built within the right of way of an active railroad.

8-6.03 Selection Guidelines

Although incorporating bicyclists' needs into the design of major transportation corridors can be challenging, the reality of planning bikeways in built environments means that roadways constitute the majority of a bicycle network. Whenever roadways are constructed or reconstructed, appropriate provisions for bicyclists should be addressed.

The bikeway design options include:

1. shared lanes
2. paved shoulders
3. bicycle lanes
4. shared use paths

Bicycle routes are not included in the list above because they represent a designation rather than a facility type.

The best application of each of these facilities combines experience with data analysis, engineering judgment, and budget constraints.

Selection of an appropriate bikeway facility should be based on the following information:

1. roadway function (arterial, collector, local)
2. traffic volume
3. speed
4. traffic mix (e.g., truck percent)
5. expected users (e.g., is one type of user expected to dominate, such as children bicycling to school)
6. road conditions (lane widths, total roadway width, conditions at intersections, parking demand)
7. frequency of driveways and side roadways
8. topography
9. existing and proposed adjacent land uses

Table 8-6-A outlines general considerations for each facility type.

Table 8-6-A
GENERAL CONSIDERATIONS FOR BIKEWAYS

Type Of Bikeway	Best Use	Motor Vehicle Design Speed	Traffic Volume	Classification or Intended Use	Other Considerations
Paved Shoulders	Rural roadways that connect town centers and other major attractors.	Variable. Typical posted rural roadway speeds (generally 40 mph – 55 mph)	Variable	Rural roadways; inter-city roadways	Provides more shoulder width for roadway stability. Shoulder width is dependent upon characteristics of the adjacent motor vehicle traffic (e.g., wider shoulders on higher-speed roads).
Bicycle Lanes	Major roadways that provide direct, convenient, quick access to major land uses. Also can be used on collectors and busy urban roadways with slower speeds.	Generally, any roadway where the design speed is more than 25 mph.	Variable speed differential is generally a more important factor in the decision to provide bicycle lanes than traffic volumes.	Arterials and collectors intended for major motor vehicle traffic movements.	Where motor vehicles are allowed to park adjacent to bicycle lane, the width of bicycle lane should be sufficient to reduce probability of conflicts due to opening vehicle doors and other hazards. Intersections should be analyzed to reduce bicyclist/motor vehicle conflicts. Sometimes bicycle lanes are left “undesignated” (i.e., bicycle symbol and signs are not used) in urban areas as an interim measure.
Shared Lanes (wide outside lanes)	Major roadways where bicycle lanes are not selected due to space constraints or other limitations.	Variable. Used as the speed differential between bicyclists and drivers increases. Generally any roadway where the design speed is more than 25 mph.	Generally more than 3000 vehicles per day	Arterials and collectors intended for major motor vehicle traffic movements.	Opportunities to provide parallel facilities should be explored for less confident bicyclists.

Table 8-6-A
GENERAL CONSIDERATIONS FOR BIKEWAYS (Continued)

Type of Bikeway	Best Use	Motor Vehicle Design Speed	Traffic Volume	Classification or Intended Use	Other Considerations
Shared Lanes (shared lane markings)	Space-con-strained roadways with narrow travel lanes, or roadway segments upon which bicycle lanes are not selected due to space constraints or other limitations.	Variable. Used where the speed limit is 35 mph or less.	Variable. Useful where there is high turnover in on-street parking to prevent crashes with open car doors.	Collectors or minor arterials	May be used in conjunction with wide outside lanes. Opportunities to provide parallel facilities should be explored for less confident bicyclists. Where motor vehicles are allowed to park along shared lanes, marking placement should be evaluated to reduce potential conflicts with opening car doors.
Shared Roadways (no special provisions)	Crossing routes with low speeds and volumes where bicycles can share the roadway with no special provisions.	Speed differential between drivers and bicyclists is typically 15 mph or less. Generally, speed limits of 30 mph or less.	Generally less than 1000 vehicles per day	Neighborhood or local roadways	Can provide an alternative to busier streets in a gridded street network. On a non-grid network, may be circuitous or discontinuous.
Shared Use Path: independent corridor	Linear corridors in greenways, or along waterways, roadways, active or abandoned rail lines, utility rights of way, and unused rights of way. May be a short connection, (e.g., pathway connector between two cul de sacs) or a longer connection.	n/a	n/a	Provides a separated path for non-motorized users.	Intersections should be analyzed to anticipate and mitigate conflicts between path and roadway users. Path should be designed considering all users and wide enough to accommodate expected usage. On-road alternatives may be desired for advanced riders who desire a more direct facility that accommodates higher speeds.

8-6.04 Design Criteria

For the design criteria of bicycle facilities, AASHTO's *Guide for the Development of Bicycle Facilities* (Bike Guide) should be referenced. The following information is a summary of the *Bike Guide* concerning recommended widths of bicycle facilities:

1. Shared Lanes on Major Roadways – Travel lane widths of 14 feet to 16 feet are desirable. Lane widths that exceed 16 feet are not recommended as they may encourage two motor vehicles to travel side by side.

2. Paved Shoulders – Paved shoulders may be used on rural roadways to accommodate bicycle travel, with or without the use of specific pavement markings or signs. For non-curbed sections with no vertical obstructions immediately adjacent to the roadway, a minimum 4-foot paved shoulder should be provided. If bicyclists are expected to be traveling adjacent to guardrail, curb, or other roadside barriers, a minimum 5-foot paved shoulder should be provided. In addition, wider shoulders should be considered where design speeds exceed 50 miles per hour, or where the volume of trucks, buses, or recreational vehicles is significant.

3. Bicycle Lanes – Bicycle lanes are a portion of a roadway that has been designated by striping and/or signing for preferential or exclusive use by bicyclists. When proposed on 2-way roadways, bicycle lanes should be provided on both sides of the roadway. The following should be considered when determining the recommended bicycle lane widths:

    a. On urban sections with curbs, a minimum 4-foot bicycle lane width should be provided. A minimum operating width of four feet should be provided that does not include gutter or drainage inlets. When retrofitting roads for bicycle facilities on constrained roadways, in some cases it may not be feasible to provide this recommended operating width. In such situations, a minimum three feet of operating space should be provided between the edge line and the gutter joint. The gutter pan, although not considered traversable, should still provide a buffer from the curb for safety.

    b. Where guardrail or another barrier exists immediately adjacent to the bicycle lane, a minimum 5-foot bicycle lane width should be provided.

    c. Where design speeds are 50 miles per hour or more, or where the volume of trucks, buses, or recreational vehicles is significant, a minimum 6-foot bicycle lane should be provided. A wider bicycle lane provides additional lateral separation between motor vehicles and bicycles to minimize wind blast and other effects.

    d. Adjacent to a narrow parking lane (seven feet) with high turnover (e.g., those servicing restaurants, shops, entertainment venues), a wider bicycle lane (six to seven feet) provides more operating space for bicyclists to ride beyond the area of opening vehicle doors.

4. Shared Use Paths – The minimum paved width for a 2-directional shared use path is 10 feet, with typical widths ranging from 10 to 14 feet. A wider path is needed to provide an acceptable LOS on pathways that are to be frequently used by both pedestrians and

wheeled users. In rare circumstances, a reduced width of eight feet may be used for a short distance due to a physical constraint or at locations where bicycle and pedestrian traffic is expected to be occasional.

A wide separation should be provided between a 2-way shared use path and the adjacent roadway to demonstrate to both the bicyclist and the motorist that the path functions as an independent facility. The minimum distance between a path and the roadway curb or edge of traveled way should be five feet. Where a separation of less than five feet is proposed, a physical barrier or railing should be provided between the path and the roadway.

8-7.0 LIGHTING

AASHTO's *Roadway Lighting Design Guide* presents generally accepted warranting conditions for installing fixed roadway lighting, which are applicable in Mississippi. Additional information is available in NCHRP Report No. 152 *Warrants for Highway Lighting* and the FHWA *Roadway Lighting Handbook*.

Roadway lighting should be considered at the following locations:

1. system interchanges (between interstate routes)
2. interchanges where the total current ADT ramp traffic entering and leaving the freeway within the interchange area exceeds 10,000 for urban conditions, 8000 for suburban conditions, or 5000 for rural conditions
3. sections of divided roadway with median widths of less than 16 feet
4. sections of roadways with six or more through travel lanes
5. sections of roadways approaching illuminated interchange areas to safely transition drivers from unlighted to lighted areas
6. rest areas, hospitality stations, and weigh stations (including entrance and exit ramps and the site area)
7. locations experiencing a larger number of nighttime crashes than daytime crashes

Prior to lighting being installed on a facility, the Department and the local agency typically sign an agreement defining the responsibilities of each agency for the cost of the power and who is responsible for the maintenance of the lighting system.

The lighting designer should design the lighting layout, prepare the lighting plans, and recommend specifications, while coordinating with the District, Roadway Design Division, Bridge Design Division, Geotechnical Branch, and Construction Division as applicable.

A lighting layout should be submitted to the Bridge Design Division for bridges where future lighting is expected so that brackets for messenger cable, lighting assembly mounts, and anchor bolts may be provided as part of the bridge structure

8-8.0 REFERENCES

1. *Guide for Transportation Landscape and Environmental Design*, AASHTO. 1991.

2. *ADA Accessibility Guidelines*, US Access Board, 2006.

3. Draft *Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way*, US Access Board, 2011.

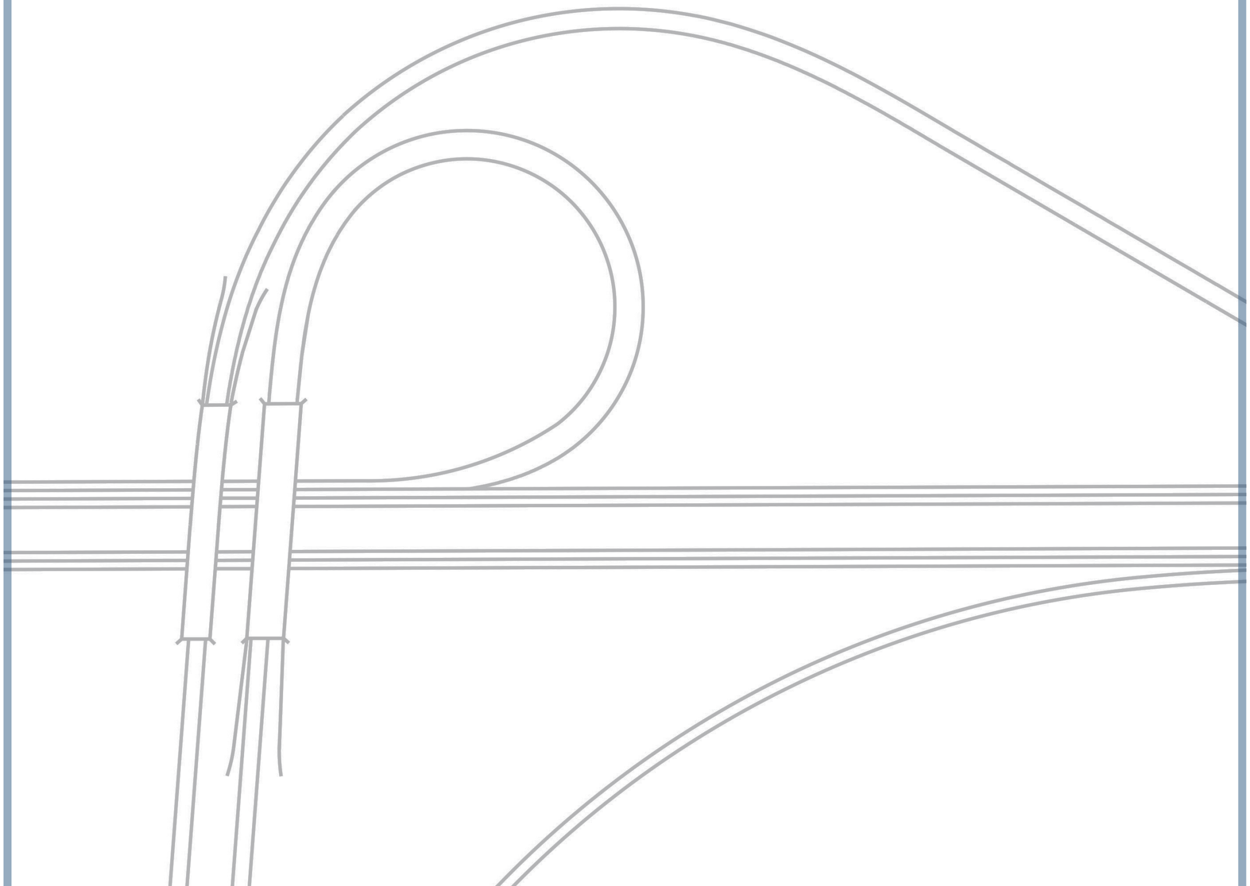
4. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2018.

5. Procedures for Handling Airspace Matters, JO 7400.2J, Federal Aviation Administration, 2012.

6. *Safe, Efficient Use and Preservation of Navigable Airspace*, 14 CFR 77, Code of Federal Regulations.

7. *A Guide for the Development of Rest Areas on Major Arterials and Freeways*, AASHTO, 2001.

8. *Guide for the Development of Bicycle Facilities*, AASHTO, 2012.



CHAPTER 9

Roadside Safety

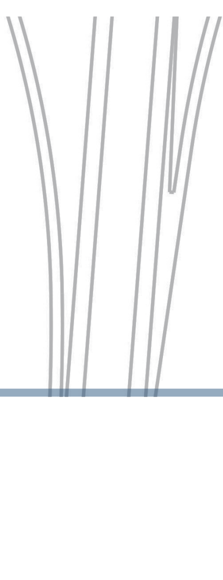


Table of Contents

Chapter 9
ROADSIDE SAFETY

Roadside safety can be considered the design of the area beyond the limits of the traveled way. Roadside safety features should be implemented to the extent feasible to realize the maximum safety benefit for each unique set of circumstances. The information presented in this chapter is mostly applicable to new construction or major reconstruction projects, but any roadside safety improvements along existing roadways using 3R criteria should be evaluated to gain the maximum benefit with available funds. Chapter 12, "Existing Roadways", provides more information for clear zone criteria for projects on existing roadways.

9-1.0 DEFINITIONS/NOMENCLATURE

1. Barrier Warrant – A criterion that identifies an area of concern that should be shielded by a barrier, if judged to be feasible.

2. Backslope – The side slope created by connecting the ditch bottom or roadside shelf (see Section 14-2.06), upward and outward, to the natural ground line.

3. Barrier – A device that provides a physical limitation through which a vehicle cannot normally pass, and which is intended to contain or redirect an out-of-control vehicle away from obstructions or opposing traffic.

4. Barrier Terminals – End treatments for roadside barriers, median barriers, and transitions to other types of barriers (e.g., to bridge rails).

5. Bi-Directional Barrier – A barrier designed to safely handle an impact from either direction of traffic. A bi-directional barrier can redirect vehicles striking its face from one side or both sides.

6. Cable Barrier – See Flexible Barrier.

7. Clear Zone – The total roadside border area, starting at the edge of the traveled way, available for safe use by out-of-control vehicles. This area may consist of a shoulder, recoverable slope (i.e., a slope of 4:1 or flatter), non-recoverable slope, and/or clear runout area free of any non-crashworthy obstacles. The desired width of the clear zone is dependent upon traffic volume, speed, and roadside geometry.

8. Concrete Barrier – A rigid barrier (commonly referred to as concrete median barrier) that can withstand most vehicular impacts without penetration.

9. Crash Cushion – A device that prevents an out-of-control vehicle from impacting fixed objects by gradually decelerating the vehicle to a safe stop or by redirecting the vehicle away from the obstacle. This type of end treatment is also known as an impact attenuator.

10. Crashworthy – A feature that has been proven acceptable for use under specified conditions through crash testing or in-service performance.

11. Guardrail End Treatment – The crashworthy modification of the end of a guardrail.

    a. End Treatment (Leading) – Generally, a crashworthy portion of guardrail that is recommended when the nearest end of rail is located within the clear zone.

    b. End Treatment (Trailing) – Generally, a portion of guardrail that may only be used when the farthest end of rail is located outside the clear zone for the opposing direction of traffic.

12. Fill Slope – Slope extending outward and downward from the shoulder hinge point to intersect the natural ground line.

13. Flare Rate – The rate of lateral change at which a longitudinal barrier may be installed relative to its longitudinal length.

14. Flexible Barrier – A longitudinal barrier that deflects significantly upon impact. Energy is dissipated by the barrier deflection with less vehicular deceleration.

15. Foreslope – The side slope in a cut section created by connecting the shoulder, downward and outward, to the ditch bottom.

16. Functionally Obsolete Guardrail – Guardrail installed under previous standards that does not meet criteria for new standards, but does meet an acceptable previous performance level and may be maintained until it is damaged beyond repair.

17. Gating – A term used to describe guardrail end treatments that are designed to allow controlled penetration by an impacting vehicle. Gating portions of end treatments do not shield hazards or obstructions and should not be considered as part of the length of need calculation.

18. Impact Angle – For a longitudinal barrier, the angle between a tangent to the face of the barrier and a tangent to the vehicle's path at impact. For a crash cushion, an impact angle is the angle between the axis of symmetry of the crash cushion and a tangent to the vehicular path at impact.

19. Impact Attenuator – See Crash Cushion.

20. Length of Need – Total length of a longitudinal barrier needed to shield an area of concern, measured with respect to the centerline of roadway. The length of need is measured to the last point of full-strength rail. Length of need is a function of runout length, the location of the hazard or obstruction, and the location of the barrier installation. See Figure 9-5-A.

21. Longitudinal Barrier – A flexible, semi-rigid, or rigid barrier whose primary function is to prevent penetration and to safely redirect an out-of-control vehicle away from a roadside or median obstacle.

22. *Manual for Assessing Safety Hardware (MASH)* – The current national publication prepared by AASHTO to determine the performance level of roadside safety hardware. *MASH* supersedes NCHRP 350. *MASH* presents uniform guidelines for the crash testing

of both permanent and temporary roadway safety features. Recommended evaluation criteria is also provided to assess test results.

23. Median Barrier – A longitudinal barrier used to prevent an out-of-control vehicle from crossing the median of a divided roadway, preventing collisions between traffic traveling in opposite directions.

24. Non-Recoverable Parallel Slope – Slope that can be safely traversed, but upon which an out-of-control vehicle is unlikely to recover. An out-of-control vehicle will likely continue down to the toe of slope. If a fill slope is between 3:1 (inclusive) and 4:1 (exclusive), it is considered a non-recoverable parallel slope.

25. Non-Traversable Slope – Slope steeper than 3:1 that is generally considered non-traversable and may warrant shielding if within the clear zone.

26. Operational System – A roadside barrier, end terminal, or crash cushion that has performed satisfactorily in full-scale crash tests and has demonstrated satisfactory in-service performance.

27. Parallel Slopes – Cut and fill slopes for which the toe of slope runs approximately parallel to the flow of traffic.

28. Recoverable Parallel Slope – A slope on which a driver can likely retain or regain control of the vehicle. Slopes equal to or flatter than 4:1 are considered recoverable.

29. Rigid Barrier – A longitudinal barrier that does not deflect upon impact.

30. Roadside Barrier – A longitudinal barrier used to shield hazards located within an established clear zone. Roadside barriers include guardrail, concrete barriers, flexible barriers, etc.

31. Roadside Clear Zones – See Clear Zone.

32. Roadside Hazards – A general term to describe roadside features that cannot be safely impacted by an out-of-control vehicle. Roadside hazards include both fixed objects and non-traversable roadside features. See Section 9-3.02.

33. Rub Rail – Additional rail member placed below standard rail to provide additional stiffness and to reduce the gap between the rail and the ground surface.

34. Runout Length (Barrier Design) – Estimated distance for stopping an out-of-control vehicle. Design runout length is a function of design speed and traffic volume. Runout length is not to be confused with the tangent runout length recommended for superelevation transitions.

35. Safety Shape – A concrete barrier with slopes on the exposed face(s) used to redirect vehicles upon impact.

36. Safety Slope – See Recoverable Parallel Slope.

37. <u>Semi-Rigid Barrier</u> – A longitudinal barrier ranging from almost rigid to flexible.

38. <u>Shy Distance</u> – The distance from the edge of traveled way beyond which a roadside object should not be perceived as an immediate hazard by the typical driver, to the extent that the driver may change vehicular placement or speed.

39. <u>Side Slope</u> – A ratio used to express the steepness of a slope adjacent to the roadway. The ratio is expressed as horizontal to vertical (H:V).

40. <u>Terminal</u> – A crashworthy anchorage used to anchor a flexible or semi-rigid barrier to the ground. Being crashworthy, terminals are normally used at the end of a barrier that is located within the clear zone or is likely to be impacted by out-of-control vehicles.

41. <u>Test Level</u> – An AASHTO measurement for evaluating crash performance of roadside components.

42. <u>Toe of Slope</u> – The intersection of the fill slope with the natural ground line.

43. <u>Top of Slope</u> – The intersection of the cut backslope with the natural ground line.

44. <u>Transverse (or Intersecting) Slopes</u> – Cut and fill slopes for which the toe/top of slope runs approximately perpendicular to the flow of traffic. Transverse slopes or intersecting slopes are typically formed by intersections between the mainline and median crossovers, berms, driveways, or side roads.

45. <u>Traversable Slope</u> – A slope from which a driver will be unlikely to steer back to the roadway, but may be able to slow and stop safely. Slopes between 3:1 (inclusive) and 4:1 (exclusive) generally are considered traversable if they are smooth and free of obstructions.

46. <u>Uni-Directional Barrier</u> – A barrier designed to safely handle an impact from only one direction, from front to rear.

9-2.0 CLEAR ZONES

9-2.01 <u>General</u>

The clear zone widths presented in this manual should be placed in proper perspective. The distances imply a degree of accuracy that does not exist. They do, however, provide a good frame of reference for making decisions on providing a roadside area that is as safe as possible. Each application of the clear zone distance should be evaluated individually and good engineering judgment exercised. In general, as much clear zone as can be obtained feasibly should be provided. When using the recommended clear zone distances, the following information should be applied:

1. <u>Project Scope of Work</u> – The clear zone distances in Table 9-2-A should be applied to all freeway projects (new construction/reconstruction and 3R) and to new construction/reconstruction projects on non-freeways. Desirably, these distances should also be considered for use on projects on existing non-freeways for which 3R criteria is

being used. However, if their use on existing non-freeways is considered infeasible, Chapter 12, "Existing Roadways", should be referenced.

2. Context – As a general statement, the use of an appropriate clear zone distance is a compromise between maximum safety and minimum construction costs. If a formidable obstacle lies just beyond the clear zone, removing or shielding the obstacle may be appropriate if costs are reasonable. Conversely, cases may arise where the clear zone should not be achieved at all costs. Limited right of way or unreasonable construction costs may lead to installation of a barrier.

3. Roadside Cross Section – The recommended clear zone distance should be based on the type of roadside cross section. Sections 9-2.02 and 9-2.03 present schematics for various cross sections.

4. Measurement – Clear zone distances should typically be measured from the edge of traveled way, except where a long (greater than 0.5 miles) auxiliary lane exists. See Section 9-2.02 for more information on measuring clear zone distances based on the steepness of the slope. See Section 9-2.05 for more information about clear zones where auxiliary lanes are present.

9-2.02 Fill Slopes (Non-Curbed)

Table 9-2-A presents the Department's recommended clear zone distances for non-curbed roadways. Figure 9-2-A illustrates the types of fill slopes. The following points explain the application of Table 9-2-A for various fill slopes:

1. Recoverable Fill Slopes – For parallel fill slopes 4:1 and flatter (Figure 9-2-A(a)), the recommended clear zone distance can be determined directly from Table 9-2-A.

2. Non-Recoverable Fill Slopes – For parallel fill slopes between 3:1 (inclusive) and 4:1 (exclusive) (Figure 9-2-A(b)), the recommended recovery area includes a distance beyond the toe of slope, also referred to as the clear runout area, that compensates for the clear zone distance not provided along the non-recoverable fill slope. Sufficient right of way should be purchased to ensure that the area can be maintained and cleared of hazards.

   The clear runout area should be 10 feet or the distance determined from the 6:1 or flatter column of Table 9-2-A for the applicable design speed and traffic volume, whichever is greater. The slope of the clear runout area should desirably be 6:1 or flatter.

3. Barn-Roof Fill Slope (Recoverable/Non-Recoverable) – Barn-roof fill slopes may be designed with a recoverable slope (or safety slope) leading to a non-recoverable slope (Figure 9-2-A(c)). The clear zone should typically be provided entirely on the recoverable slope (i.e., the combined width of the shoulder and the recoverable slope should equal or exceed the clear zone distance). The slope after the break is typically 3:1. If the clear zone based on the recoverable slope extends beyond the slope break between the recoverable and non-recoverable slope, the desirable 6:1 or flatter clear runout area should be 10 feet or the remainder of the clear zone distance located on the non-recoverable slope, whichever is greater.

4. Non-Traversable Fill Slope – Fill slopes steeper than 3:1 are considered non-traversable (Figure 9-2-A(d)). Non-traversable fill slopes that remain within the clear zone typically warrant a barrier, in which case the recommended clear zone would not be provided. Section 9-3.0 should be referenced for barrier warrants.

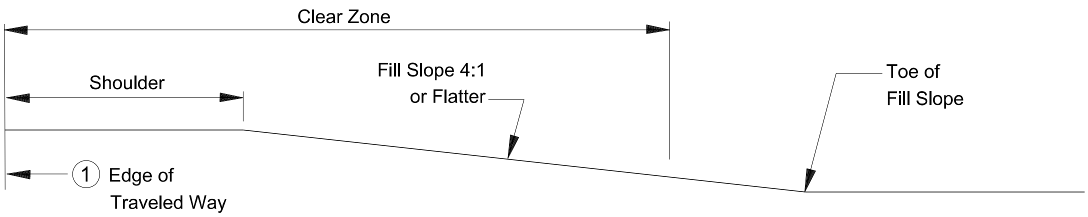
Table 9-2-A
RECOMMENDED CLEAR ZONE DISTANCES (feet)
(Non-Curbed Facilities)

Design Speed (mph)	Design ADT	Fill Slope/Cut Foreslope		
		Recoverable Safety Slopes (Figure 9-2-A(a))		Non-recoverable (Figure 9-2-A(b)) (Steeper than 4:1 and 3:1 or flatter)
		6:1 or Flatter	5:1 to 4:1	
40 or Less	Under 750	7 – 10	7 – 10	SEE SECTION 9-2.02.
	750 – 1500	10 – 12	12 – 14	
	1501 – 6000	12 – 14	14 – 16	
	Over 6000	14 – 16	16 – 18	
45 – 50	Under 750	10 – 12	12 – 14	
	750 – 1500	14 – 16	16 – 20	
	1501 – 6000	16 – 18	20 – 26	
	Over 6000	20 – 22	24 – 28	
55	Under 750	12 – 14	14 – 18	
	750 – 1500	16 – 18	20 – 24	
	1501 – 6000	20 – 22	24 – 30	
	Over 6000	22 – 24	26 – 32*	
60	Under 750	16 – 18	20 – 24	
	750 – 1500	20 – 24	26 – 32*	
	1501 – 6000	26 – 30	32 – 40*	
	Over 6000	30 – 32*	36 – 44*	
65 – 70	Under 750	18 – 20	20 – 26	
	750 – 1500	24 – 26	28 – 36*	
	1501 – 6000	28 – 32*	34 – 42*	
	Over 6000	30 – 34	38 – 46*	

* *Clear zone distances may be limited to 30 feet for feasibility and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance.*

Notes:

1. *This table applies to all freeways and new construction/reconstruction projects on non-freeways.*
2. *All distances for recoverable safety slopes are measured from the edge of traveled way or from the edge of an auxiliary lane (if the auxiliary lane is longer than 0.5 miles). See Figure 9-2-A for clear zone recommendations on non-recoverable slopes.*
3. *The Design ADT should be the total ADT for both directions of travel for the design year used for the project.*
4. *On interchange ramps, the clear zone on each side of the ramp should be equal to the clear zone on the mainline, except for the portion of the ramp near a controlled ramp terminal. See Section 9-2.06 for more information.*

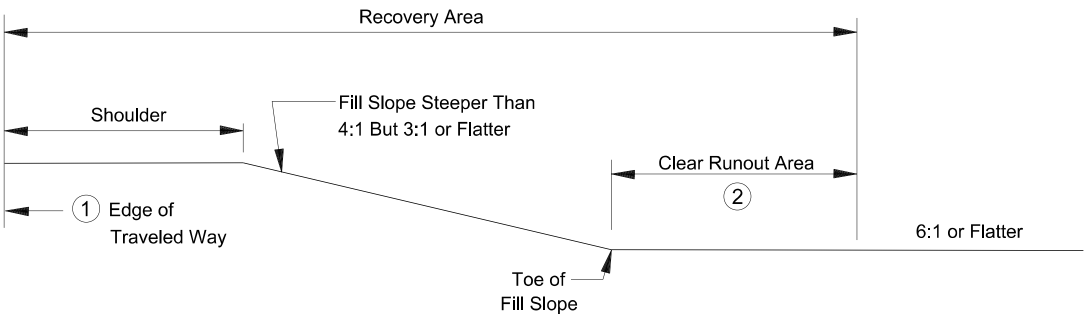


(a) Recoverable Parallel Fill Slope

Notes:

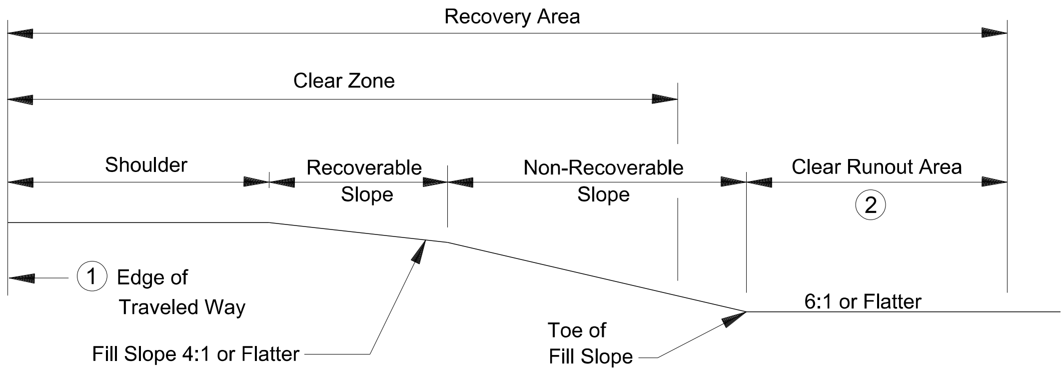
1. In some cases, the edge of the auxiliary lane. See Section 9-2.05.

2. 10' or clear zone value from Table 9-2-A, whichever is greater.



(b) Non-Recoverable Parallel Fill Slope

CLEAR ZONE APPLICATION FOR PARALLEL FILL SLOPES
(Non-Curbed)
Figure 9-2-A

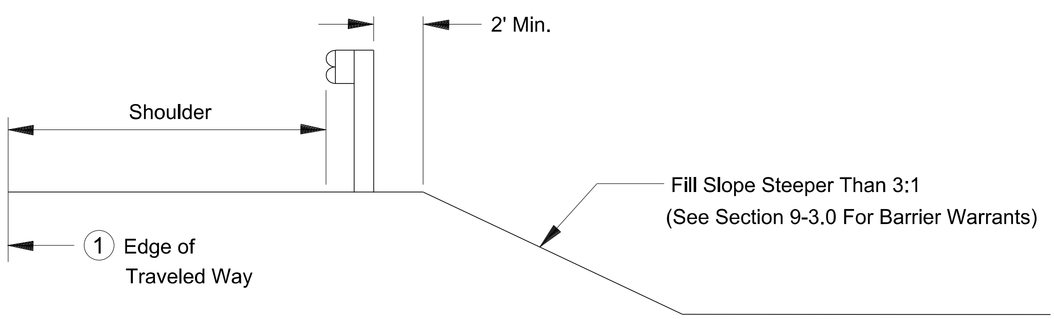


(c) Barn-Roof Parallel Fill Slope
(Recoverable/Non-Recoverable)

Notes:

1. In some cases, the edge of the auxiliary lane. See Section 9-2.05.

2. 10' or remainder of clear zone value from Table 9-2-A on non-recoverable slope, whichever is greater.



(d) Non-Traversable Parallel Fill Slope

CLEAR ZONE APPLICATION FOR PARALLEL FILL SLOPES
(Non-Curbed)
Figure 9-2-A (Continued)

9-2.03 Cut Slopes (Non-Curbed)

Cut ditch sections, as illustrated in Figure 9-2-B, are typically constructed in roadside cuts without curbs. The bottom of the ditch should be located outside of the clear zone on new construction/reconstruction projects, as well as on 3R projects, if feasible.

The following procedures should be used to determine the recommended clear zone distance:

1. Foreslope – Table 9-2-A should be used to determine the clear zone based on the foreslope.

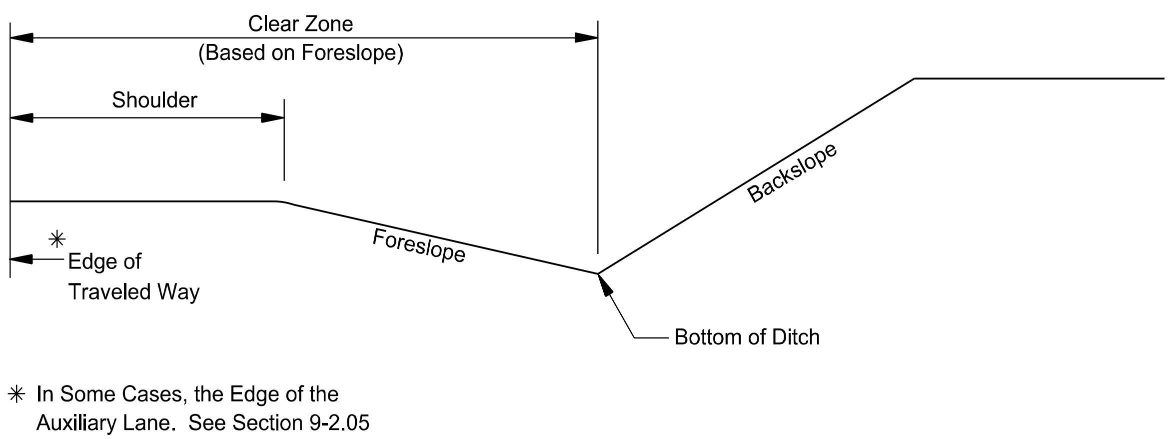
2. Location of the Ditch – The bottom of the ditch should be checked to determine if it is within the clear zone. If not, then only roadside hazards within the clear zone on the foreslope usually need to be considered. For ditches within the clear zone, see Item 3 below.

3. Ditch Within the Clear Zone – The feasibility of modifying or relocating the ditch using one or more of the following methods should be considered:

   a. The cut foreslope may be flattened to reduce the recommended clear zone width. However, the designer should ensure that the ditch has an adequate depth and slope to facilitate sufficient drainage.

   b. The cut foreslope may be lengthened to move the ditch outside of the clear zone. However, this approach would result in a deeper ditch that could cause undesirable right of way impacts.

   c. If it is infeasible to move the ditch outside of the clear zone, the feasibility of using a preferred channel cross section as discussed in AASHTO's *Roadside Design Guide* should be analyzed.



CLEAR ZONE APPLICATION FOR CUT SLOPES
(Non-Curbed)
Figure 9-2-B

9-2.04 Curbs

The clear zone criteria presented in Table 9-2-A apply to all rural roadways, whether curbed or non-curbed. Curbs should not be used on rural facilities except along the turning radii at intersections. Section 14-2.06.09 provides additional information concerning clear zones and lateral offsets on urban roadways.

9-2.05 Auxiliary Lanes

For auxiliary lanes greater than 0.5 miles, the clear zone should be measured from the outside edge of the auxiliary lane. Where turning lanes are located at intersections, the clear zone should be measured from the edge of the traveled way. However, for urban areas where the 1.5-foot lateral offset applies, the offset should be measured from the face of the curb even if an auxiliary lane is present.

9-2.06 Interchange Ramps

Where feasible, the clear zone for interchange ramps should equal the clear zone on the roadway mainline. Where it is infeasible to match the mainline clear zone, the ramp clear zone should be based on the design speed, traffic volume, horizontal curvature, and roadside geometry on the ramp. If a ramp is terminated at an intersection with stop or signal control, clear zones in Table 9-2-A are not applicable to the portion of the ramp near the intersection. The clear zone should be measured from the edge of the traveled way on both sides of the ramp.

9-3.0 ROADSIDE BARRIER WARRANTS

9-3.01 Range of Treatments

If a roadside hazard is within the clear zone, the treatment that is judged to be the most feasible for the site conditions should be selected. The range of treatments listed in order of preference includes:

1. eliminating the hazard
2. relocating the hazard outside of the clear zone
3. reducing the impact severity by using a breakaway support (sign posts, luminaire supports)
4. shielding the hazard with a roadside barrier or crash cushion
5. delineating the hazard

The primary purpose of a roadside barrier is to reduce the probability of an out-of-control vehicle striking a hazard that is less forgiving than the barrier itself. If removal, relocation, or modification of the hazard is not feasible, a barrier should be installed only if engineering judgment indicates it is a feasible solution. The selected treatment should be based upon traffic volumes, roadway geometry, proximity of the hazard to the traveled way, nature of the hazard, costs for remedial action, crash pattern, and judgment.

9-3.02 Roadside Hazards

Roadside hazards include, but are not limited to:

1. non-breakaway sign and luminaire supports
2. traffic and railroad signal poles
3. concrete bases extending more than four inches above the ground
4. bridge piers and abutments at underpasses
5. bridge parapet ends
6. retaining walls
7. trees with diameter greater than four inches (at maturity)
8. non-traversable slopes (i.e., embankments)
9. streams or permanent bodies of water
10. cross culverts with single round pipes with diameters greater than 36 inches or with multiple round pipes each with diameters greater than 30 inches
11. parallel culverts with single round pipes with diameters greater than 24 inches
12. protruding concrete headwalls on cross drains and side drains, regardless of size and opening
13. stone fences
14. utility poles and other wood poles with cross sectional area greater than 50 square inches
15. mailboxes
16. steep transverse slopes

Shielding obstacles located outside the clear zone may be appropriate, especially for features installed by the Department or sites that have a crash pattern. For example, shielding a bridge pier located outside the clear zone may be justified due to the potential severity of a crash.

9-3.03 Embankments (Fill Sections)

A barrier is typically provided for fill slopes within the clear zone that are steeper than 3:1. A barrier is not warranted on fill slopes 3:1 or flatter unless roadside hazards exist within the clear zone as calculated from Table 9-2-A.

9-3.04 Bridge Rails

Barrier protection should be provided on all approach ends to bridge rails. A roadside barrier is typically not warranted on the trailing end of a 1-way bridge unless a barrier is necessary for other reasons (e.g., fill slope steeper than 3:1, narrow median with bridge end within clear zone of opposing traffic, temporary guardrail needed for temporary 2-way traffic on bridge). Section 9-7.0 provides information on the barrier transitions into bridge rails.

9-3.05 Abutments/Piers

Abutments and piers located within a distance of 30 feet to the edge of traveled way should be investigated for potential collisions, which should be addressed by either providing structural resistance or by redirecting or absorbing the collision load.

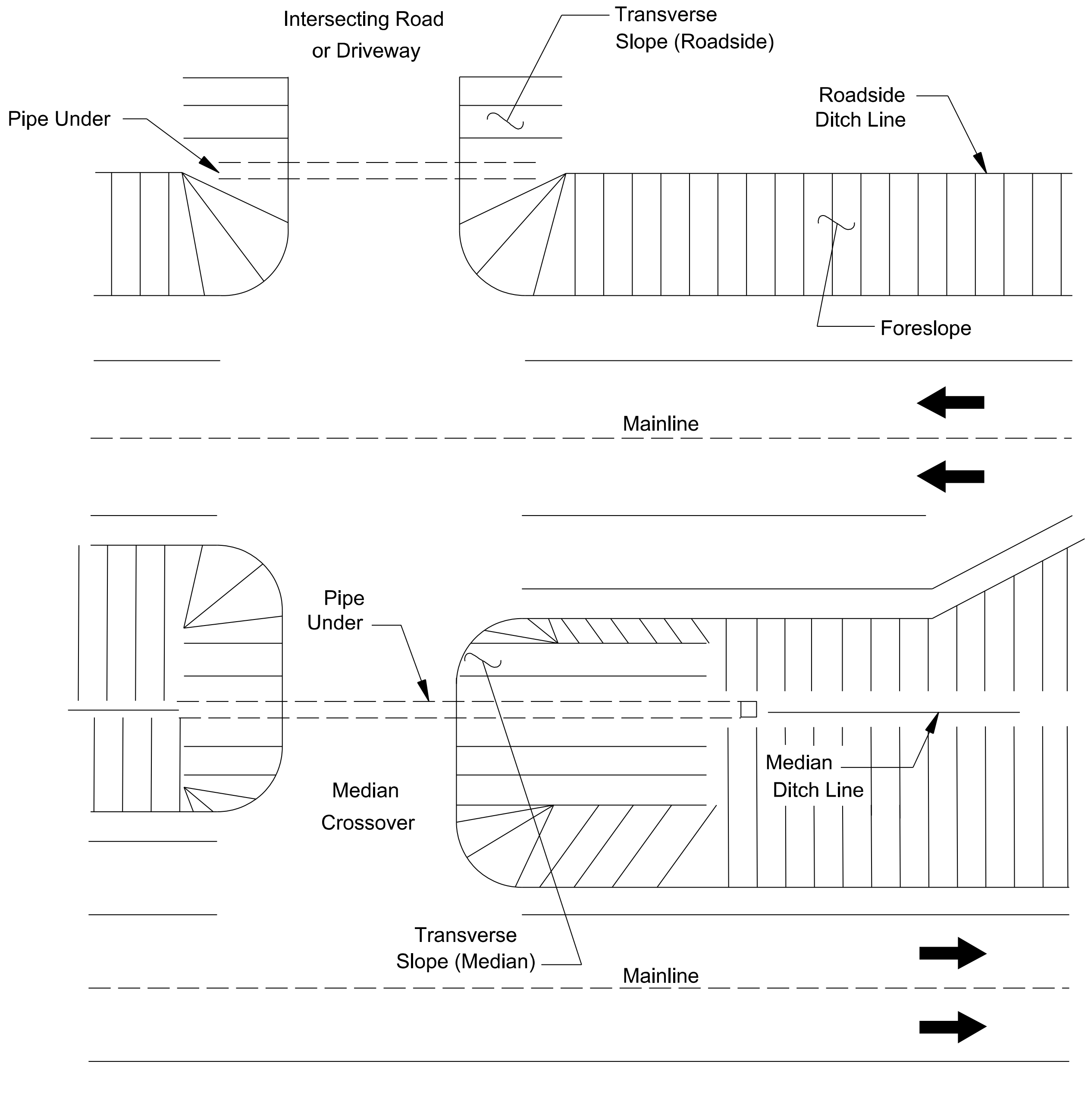
Where the decision is to provide structural resistance and the bridge is to be designed according to AASHTO's *LRFD Bridge Design Specifications*, analysis and design of the barrier should be performed following Load and Resistance Factor Design (LRFD*)* Specifications or any other applicable specifications/guidance published by AASHTO. The more stringent criteria should govern. Where the decision is to redirect or absorb the collision load, protection should be provided as illustrated in Figure 7-1-B. See the *Standard Drawings* for pier protection details.

9-3.06 Transverse Slopes

A roadway may intersect a driveway, side road, or median crossover, presenting a slope that could be impacted at an approximate 90-degree angle by out-of-control vehicles from the mainline. See Figure 9-3-A. Therefore, transverse slopes should be as flat as feasible as described below:

1. Medians ($V \geq 50$ miles per hour) – For depressed medians on multilane roadways (rural and urban), transverse slopes should be 10:1 or flatter. For drainage pipes that pass beneath a median crossover (or other parallel drains) a sufficient length of pipe should be used to ensure that the pipe ends extend to provide for the recommended transverse slopes.

2. Rural Facilities ($V \geq 50$ miles per hour) – Except for medians (see Item 1 above), transverse slopes within the clear zone should be 6:1 or flatter; however, 10:1 transverse slopes are desirable where feasible.



TRANSVERSE SLOPES
Figure 9-3-A

3. Rural Facilities (V ≤ 45 miles per hour) – Transverse slopes (including medians) should be 3:1 or flatter within the clear zone.

4. Urban Facilities (All Design Speeds) – Transverse slopes (including medians) should be 3:1 or flatter within the clear zone.

Parallel drainage structures may be closely spaced in urban areas because of multiple driveways and intersecting roads. In such locations, converting the open ditch into a closed drainage design and backfilling the ditches between adjacent driveways may be more desirable.

On projects where a median left-turn lane is added to an existing facility, the resulting transverse slopes, foreslopes, and adjacent drainage structures should be checked to ensure that they meet the criteria presented in this chapter.

9-3.07 Cross-Drain Structures

Cross-drain structures are designed to convey water beneath the roadway and prevent overtopping of the roadway; however, these structures may represent an obstacle for out-of-control vehicles. Therefore, such structures should be designed so that they are beyond the clear zone.

When infeasible to design a cross-drain structure so that the openings are beyond the clear zone (e.g., right of way or environmental constraints):

- Box culverts and cross-drain pipes with diameters/spans greater than 36 inches should be shielded with a roadside barrier.

- Pipes with diameters/spans 36 inches or less that have flared end sections do not warrant additional treatment.

The designer should also refer to Traffic Engineering Division's Typical Signing Details for guidance on where object markers are warranted to delineate cross-drain structures.

9-3.08 Parallel Drain Structures

Parallel drain structures (often referred to as side drains) are those that are oriented parallel to the main flow of traffic, which are typically used under driveways, intersecting crossing routes, and median crossovers. As with cross-drain structures, the primary objective should be to keep the parallel drain structure outside of the mainline clear zone. The design should be coordinated with that of the surrounding transverse slope. See Section 9-3.06.

9-3.09 Traffic Signals/Luminaire Supports/Utility Poles

In general, options are limited when determining acceptable locations for the placement of roadside appurtenances such as traffic signal poles, controllers, and luminaire supports, especially in urban areas. Due to visibility requirements, limited mast-arm lengths, limited right of way, restrictive geometrics, and/or pedestrian requirements, these roadside appurtenances may be placed relatively close to the traveled way. The following information should be followed when determining their placement:

- Urban Curbed Roadways – Section 14-2.06.09 provides the recommended clear zones and minimum lateral offset for urban curbed roadways.

- Open Shoulder (Urban and Rural) Roadways – The clear zone criteria discussed in Section 9-2.0 are desirable for the placement of roadside appurtenances on open shoulder sections. However, where this criteria is not feasible, the minimum lateral offset between the edge of the traveled way and the appurtenance should be the shoulder width.

9-3.10 Retaining Walls

The following information should apply to the roadside safety aspects of retaining walls within the clear zone:

- Flare Rates – The same flare rates as those provided for a concrete barrier wall should be used.
- End Treatment – Preferably, the retaining wall should be buried in a backslope, thereby shielding its end. If infeasible, a crashworthy end treatment or crash cushion should be used.

9-4.0 ROADSIDE BARRIER TYPES

AASHTO and FHWA have created a joint implementation agreement for roadside safety hardware, which can be found on FHWA's website. The agreement provides a phased implementation plan for new permanent installations or replacements of roadside safety hardware on the National Highway System (NHS) to be *MASH*-compliant.

MASH performance crash-testing criteria apply to roadside barriers, crash cushions, end treatments, bridge rails, and guardrail-to-bridge-rail transitions. *MASH* presents six roadside barrier test levels (TL) for application to various roadway conditions based on facility type, design speed, and prevalence of heavy vehicles. The roadside safety hardware should, at a minimum, meet the TL-3 performance level whether the project is on or off the NHS.

The *Standard Drawings* provide specific details for some types of roadside safety hardware (concrete median barrier, w-beam guardrail); however, in some cases (e.g. cable barrier, guardrail terminal end sections, crash cushions), specific details should be provided by the manufacturer's specifications. The following subsections briefly describe each system and its typical usage.

9-4.01 W-Beam System

The W-beam system with heavy posts is a semi-rigid system and is the most commonly used system. The Department has approved the use of two guardrail W-beam systems, one that uses 6-inch x 8-inch wood posts, and one that uses W6 x 9.0 steel posts. Both systems meet a TL-3 performance level and typically have a 4-foot deflection distance.

The *Standard Drawings* also include an adjustable W-beam guardrail that allows for raising the height of the longitudinal rail to account for future pavement overlays. Where an existing guardrail may be adjusted, end treatments should be evaluated for replacement.

9-4.02 Thrie-Beam Guardrail

The thrie-beam guardrail is a semi-rigid system on heavy posts and has a TL-3 performance level. The deeper longitudinal member of the thrie-beam guardrail decreases the probability of an impacting vehicle overriding or underriding the guardrail. Therefore, thrie-beam guardrail is typically installed as a transition into rigid obstacles (e.g., bridge parapets) or as a barrier between the traveled way and a fixed object where the deflection distance is limited (e.g., bridge piers).

9-4.03 Concrete Barrier

Concrete barrier should be considered on the roadside around rigid objects where no deflection distance is available. The 36-inch high concrete barrier is a TL-4 system, and the 42-inch high concrete barrier is a TL-5 system. The transition rail leading into the concrete barrier should be crashworthy, or a crash cushion should be provided to shield the end of the concrete barrier.

See Section 9-3.05 for information on barrier recommendations at bridge abutments or piers.

9-4.04 Barrier Selection (New Installations)

9-4.04.01 Performance Criteria

The Department is responsible for revising its roadside safety hardware practices to comply with the current national performance criteria.

Passenger cars are the most frequent users at most locations, so guardrail is often the appropriate barrier selection. However, locations with high-traffic volumes, high-crash experience, and/or a significant volume of heavy trucks and buses may warrant a higher performance level barrier (e.g., concrete barrier).

9-4.04.02 Dynamic Deflection

The dynamic deflection should be considered when selecting an appropriate roadside barrier. AASHTO's *Roadside Design Guide* provides the deflection distances for the various systems. If no deflection distance is available, a concrete barrier should be used.

9-4.04.03 Maintenance

Another consideration in selecting the barrier type is the maintenance of the system. Although the W-beam guardrail can often sustain second hits, repair with some frequency may be required. In areas of restricted geometry, high-traffic volumes, and/or where repair creates hazardous conditions for both the repair crew and for drivers using the roadway, a rigid barrier should be considered.

9-5.0 MEDIAN BARRIERS

9-5.01 Warrants

9-5.01.01 Freeways

Figure 9-5-A presents the warrants for a median barrier based on median width and traffic volumes. Traffic volumes are based on a 20-year projection. In the areas shown as optional, the decision to use a median barrier should be based on construction and maintenance costs. A median barrier may also be warranted on wider medians if a significant number of crossover crashes have occurred.

9-5.01.02 Non-Freeways

On roadways without full access control (non-freeways), the feasibility of installing a median barrier is often limited due to median crossings for at-grade intersections. Engineering judgment should be used to evaluate whether median barrier should be considered at specific locations, such as bridge ends.

9-5.02 Types

The *Standard Drawings* provide specific details for some types of roadside safety hardware (e.g., concrete median barrier, W-beam guardrail); however, in some cases (e.g., cable barrier, guardrail terminal end sections, crash cushions) specific details should be provided by the manufacturer’s specifications. The following subsections briefly describe various median barrier systems and their typical usage. For more details, see Chapter 6 of the AASHTO *Roadside Design Guide*.

9-5.02.01 W-Beam Median Barrier

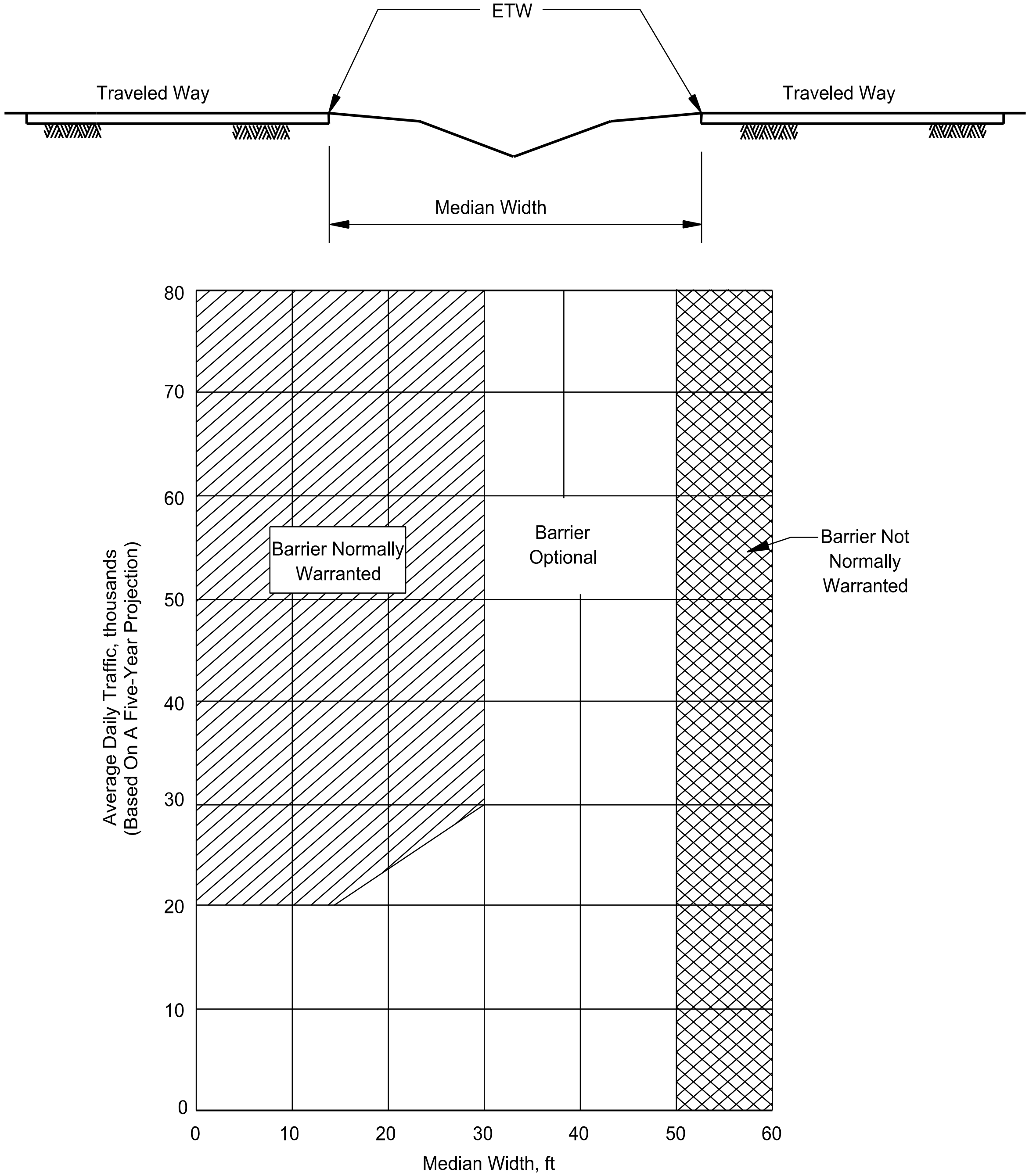
The W-beam median barrier with heavy posts is a semi-rigid system whose performance is similar to the W-beam guardrail system. When a W-beam median barrier is proposed, *Standard Drawings* for each of the two W-beam systems (wood posts and steel posts) should be included in the plans.

9-5.02.02 Thrie-Beam Median Barrier

The thrie-beam median barrier on heavy posts is also a semi-rigid system. The posts may be timber or steel. The thrie-beam median barrier’s performance is similar to the thrie-beam guardrail system. Candidate sites for using the thrie-beam median barrier are similar to those for the thrie-beam guardrail.

9-5.02.03 Concrete Median Barrier

Concrete median barrier (CMB) is typically a double-faced rigid system that does not deflect upon impact. A variation of the CMB is a half-section of the barrier system, which may be necessary where the median barrier should divide to go around a fixed object in the median (e.g., bridge pier).



MEDIAN BARRIER WARRANTS
Figure 9-5-A

9-5.02.04 Cable Median Barrier (High-Tension)

The cable median barrier is the most forgiving of the available barrier systems because of its large dynamic deflection. Most of the resistance to impact is supplied by the tensile forces developed in the cable strands. Upon impact, the cables break away from the posts, and the vehicle is able to knock down the posts as it is redirected by the cables. The deflection of these systems is typically six feet to nine feet, depending on the system, the post spacing, and the length of the barrier tested.

The lateral placement of the systems within the median should be limited to no more than four feet down the 4:1 slope for adjacent traffic impacts, and no closer than nine feet from the ditch bottom for opposite-side impacts. Cable median barrier should be placed at the shoulder edge for slopes steeper than 4:1.

The manufacturer may be contacted for detailed information on specific cable median barrier installations.

9-5.03 Median Barrier Selection

A subjective evaluation should be made and the following factors should be evaluated when selecting a median barrier system:

1. Median Width – The median width influences the probability of impact and the likely angles of impact. Higher impact angles are more likely as barrier offsets are increased. Therefore, considering both maintenance and safety, the CMB is typically the preferred barrier to be used for median widths up to approximately 30 feet, with the W-beam, thrie-beam, or cable barrier system typically used for wider medians.

2. Heavy Vehicle Traffic – A CMB is more likely to restrain and redirect heavy vehicles (e.g., trucks, buses) than the guardrail or cable barrier systems. Therefore, where a high volume of heavy vehicles is anticipated, a CMB should be considered, even on medians wider than 30 feet.

3. Costs – The initial cost of CMB often exceeds the initial cost of other median barriers. However, the maintenance costs per impact on CMB are typically far less. Considering maintenance costs, CMB is the preferred barrier for narrow medians or on high-volume roadways.

4. Maintenance Operations – The maintenance operation of repairing damaged guardrail or cable barrier systems may interrupt traffic operations, as those systems would be more likely to require lane closures for the repair. The use of CMB for narrow medians, which normally sustains far less damage when impacted, is therefore preferred for narrow medians on high-volume roadways.

9-5.04 Median Barrier Layout

The following subsections present criteria specifically for the design of median barriers. In addition, much of the information presented in Section 9-6.0 on roadside barrier layout also applies to median barriers. For example, Table 9-6-A presents the criteria for maximum flare rates that also apply to median barriers.

9-5.04.01 Divided Median Barriers

Divided median barrier may be necessary to encase or shield a fixed object in the median. Where introducing a short section of divided barrier or transitioning from a single median barrier to shield the fixed object, the slope criteria in Section 9-6.06 should be applied. Where a median barrier is divided, the adjacent transitions should not exceed the steepest recommended flare rates presented in Figure 9-6-A.

9-5.04.02 Barrier-Mounted Obstacles

If a truck or bus impacts a CMB, the high center of gravity for these vehicles may result in a vehicular roll angle that may allow the truck or bus to impact obstacles on top of the CMB (e.g., luminaire supports). If feasible, such devices should be moved to the outside of the roadway, or additional distance should be provided between the barrier and those obstacles.

9-5.04.03 Terminal Treatments

The discussion on terminal treatments in Section 9-6.07 also applies to median barriers.

9-6.0 ROADSIDE BARRIER LAYOUT

9-6.01 Length of Need

9-6.01.01 Tangent

Roadside barriers should be extended a minimum distance before a hazard to safely protect an out-of-control vehicle. Otherwise, the vehicle could travel behind the barrier and impact the hazard. Vehicles generally depart the roadway at relatively flat angles, which results in the need to extend the barrier a significant distance in advance of the hazard.

Many factors combine to determine the appropriate length of need for a given roadside condition. Table 9-6-A and Figure 9-6-A provide the information needed to calculate the barrier length of need. The basic length of need calculation is applicable to a tangent roadway alignment or where the roadside obstacle is on the inside of a horizontal curve. Examples 9-6-1 and 9-6-2 illustrate how to perform the calculations. Section 9-7.0 provides information for the length of need application to the approaching guardrail transitions into bridge rails.

Table 9-6-A
BARRIER LENGTH OF NEED

Design Speed (mph)	Design Traffic Volume (ADT)				Shy Line Offset (ft)	Steepest Flare Rate for Barrier Inside Shy Line	Steepest Flare Rate for Barrier Beyond Shy Line	
	Over 10,000	5000-10,000	1000-5000	Under 1000			Rigid (Concrete Barrier)	Semi-Rigid or Flexible (W Beam, Thrie Beam)
	Runout Length L_R (ft)	Runout Length L_R (ft)	Runout Length L_R (ft)	Runout Length L_R (ft)				
70	360	330	290	250	9.0	30:1	20:1	15:1
65	330	290	250	225	8.5	28:1	19:1	15:1
60	300	250	210	200	8.0	26:1	18:1	14:1
55	265	220	185	175	7.0	24.1	16:1	12:1
50	230	190	160	150	6.5	21:1	14:1	11:1
45	195	160	135	125	6.0	18:1	12:1	10:1
40	160	130	110	100	5.0	16:1	10:1	8:1
35	135	110	95	85	4.5	15:1	9:1	8:1
30	110	90	80	70	4.0	13:1	8:1	7:1

Equations to determine end of barrier need:

Flared Design

$$X = \frac{L_A + \frac{b}{a}(L_1) - L_2}{\frac{b}{a} + \frac{L_A}{L_R}}$$

$$Y = L_A - \frac{L_A}{L_R}(X)$$

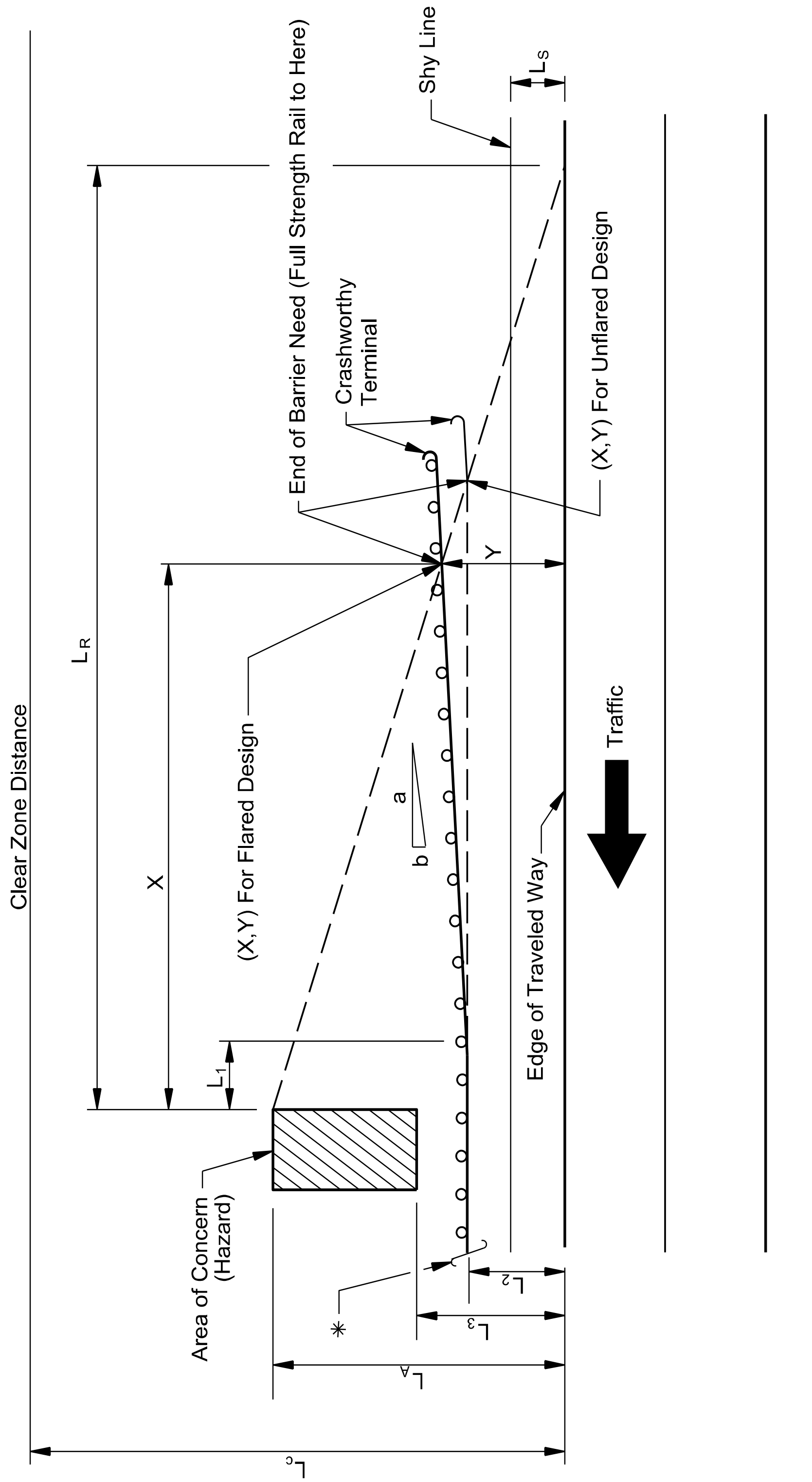
Non-flared Design

$$X = \frac{L_R\left(L_A - L_2\right)}{L_A}$$

$$Y = L_2$$

Definition of Terms:

X = length of need in advance of the area of concern
Y = lateral offset from edge of traveled way
b/a = barrier flare (e.g., 1/30)
L_c = recommended clear zone (feet)
L_A = distance from edge of traveled way to back of hazard (feet)
L_s = shy line offset, or distance at which barrier is no longer perceived as a hazard by a driver (feet)
L_R = runout length (feet)
L_1 = distance from hazard to where barrier flare begins (feet)
L_2 = distance from edge of traveled way to barrier (feet)
L_3 = distance from edge of traveled way to front of hazard (feet). ($L_3 - L_2$) should equal or exceed deflection distance.



* The distance beyond the hazard should be determined by a length-of-need calculation for opposing traffic, if applicable.

BARRIER LENGTH OF NEED
Figure 9-6-A

Example 9-6-1 (Flared Design for Guardrail at Bridge End)

Given: Design Speed = 65 miles per hour

ADT = 6000 (20-year projection)
L_A = 30 feet
L_2 = 10 feet
L_1 = 18 feet – 1¾ inches (length of standard bridge end section)
Flare Rate (a/b) = 30/1 (flared design)

Problem: First, determine the minimum length of need (X) for guardrail.

Then, determine the design length of need (X).

Solution: From Table 9-6-A, L_R = 290 feet. Therefore:

$$X = \frac{L_A + \frac{b}{a}(L_1) - L_2}{\frac{b}{a} + \frac{L_A}{L_R}}$$

$$X = \frac{30 + \frac{1}{30}(18.1458) - 10}{\frac{1}{30} + \frac{30}{290}}$$

X = (30 + 0.6049 − 10) / (0.0333 + 0.1034)
X = 20.6049 / 0.1367
X = 150.7 feet

When the barrier design is for guardrail, the actual design length of guardrail should be determined by rounding up the flared portion of the length of need ($X - L_1$) to the nearest 12.5-foot increment.

$X - L_1$ (minimum) = 150.7 − 18.1458 = 132.6 feet
$X - L_1$ (rounded up to an even 12.5-foot increment) = 11 x 12.5 feet = 137.5 feet
Design Length of Need (X) = 137.5 feet + 18.1458 feet = 155.6458 feet

Therefore, the guardrail should extend a distance of 168.1458 feet (155.6458 feet for design length of need plus 12.5 feet for gating portion of terminal end section) in advance of the leading edge of the hazard.

Example 9-6-2 (Non-Flared Design)

Given: Design Speed = 65 miles per hour
ADT = 4000 (20-year projection)
L_A = 25 feet
L_2 = 10 feet
Flare Rate (a/b) = 0 (non-flared design)

Problem: Determine the barrier length of need (X) and lateral offset (Y).

Solution: From Table 9-6-A, L_R = 250 feet. Therefore:

$$X = \frac{L_R\left(L_A - L_2\right)}{L_A}$$

$$X = \frac{250\left(25 - 10\right)}{25}$$

$$X = 150 \text{ feet}$$

$$Y = L_2$$

$$Y = 10 \text{ feet}$$

When the barrier design is for guardrail, the guardrail should extend a distance of 162.5 feet (150 feet for length of need plus 12.5 feet for gating portion of terminal end section) in advance of the leading edge of the hazard.

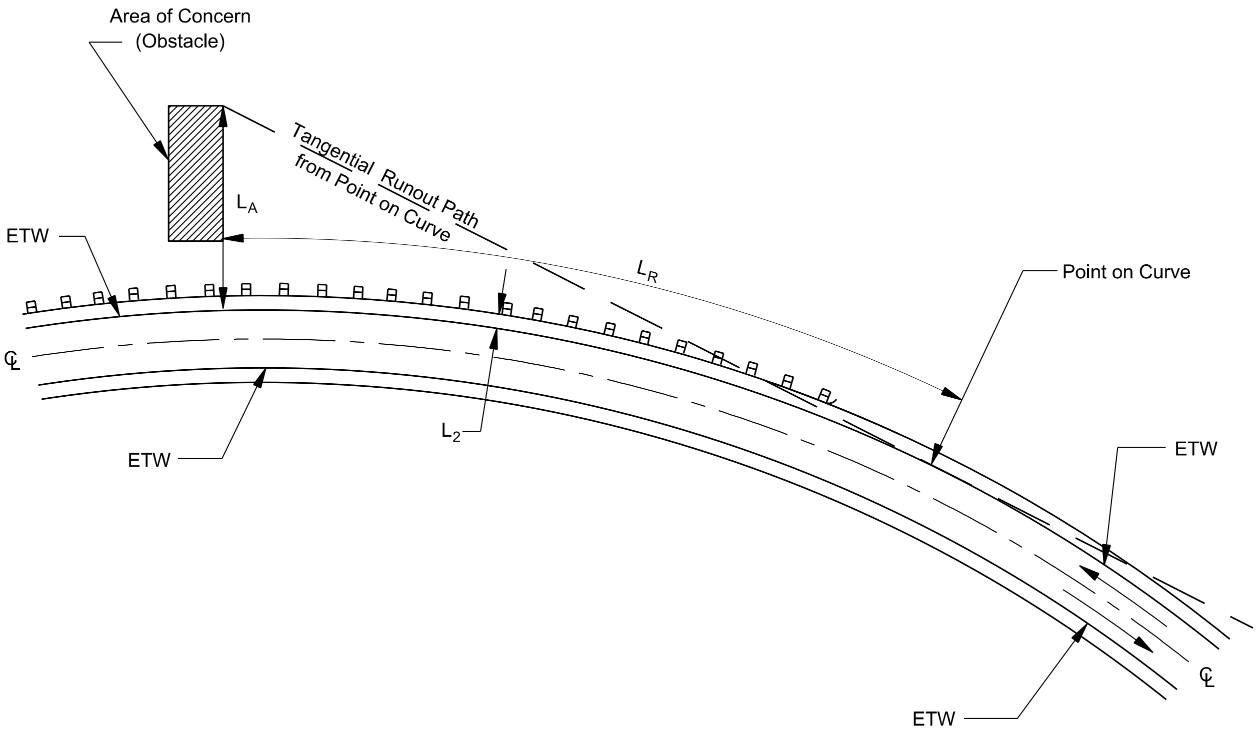
Note: If a flare rate of 15:1 had been used in advance of the hazard ($L_1 = 0$), X would have been reduced to 90 feet. Y would then be 16 feet from the edge of traveled way. However, a flared design should only be used if a 10:1 or flatter slope is present between the barrier and traveled way. See Section 9-6.06.

9-6.01.02 Outside of Horizontal Curves

A vehicle leaving the roadway on the outside of a horizontal curve will generally follow a tangential runout path. The designer should graphically measure the theoretical runout distance as shown in Figure 9-6-B. The theoretical runout distance L_R and length of need equations from Table 9-6-A for a non-flared design should be used to determine the barrier length of need outside of a horizontal curve.

9-6.02 Flare Rates

A roadside barrier is considered flared when it is not parallel to the edge of traveled way. Flared roadside barriers are normally used to locate the barrier terminal farther from the roadway, reduce the total length of guardrail needed, minimize a driver's reaction to an obstacle near the road by gradually introducing a parallel barrier installation, or to transition a roadside barrier to an obstacle closer to the roadway (e.g., bridge parapet or railing). Table 9-6-A presents the Department's criteria for barrier flare rates, which are based on the design speed, location of the barrier with respect to the shy line, and type of roadside barrier. Flatter flare rates should be provided where feasible. A standard 30:1 flare rate is typically used for guardrail at bridge ends; see Tables 9-7-A and 9-7-B.



BARRIER LENGTH OF NEED
(Outside of Horizontal Curve)
Figure 9-6-B

9-6.03 Lateral Placement

Roadside barriers should be placed as far as feasible from the edge of traveled way. Such placement provides an out-of-control driver the best chance of regaining control of the vehicle without impacting the barrier. The following factors should be considered when determining barrier lateral placement:

1. Deflection – Obstacles should not be within the dynamic deflection distance of the barrier, as measured from the back of the post.

2. Slope – The slope between the traveled way and barrier should be no steeper than 10:1. See Section 9-6.06.

3. Post Support – A 2-foot minimum space should be provided between the back of the barrier post and the outside edge of shoulder.

4. Shy Distance – Drivers tend to shy away from continuous longitudinal obstacles along the roadside (e.g., guardrail). Therefore, the minimum lateral barrier offset should be based on the shy distance criteria in Table 9-6-A.

9-6.04 Barrier Gaps

Openings or breaks in barriers should be kept to a minimum. Barrier gaps of less than 200 feet should be connected, unless a break in the barrier is needed for access (e.g., driveways, maintenance operations).

9-6.05 Placement behind Curbs

The use of curb/guardrail combinations should preferably be avoided where high-speed, high-angle impacts are likely. Possible alternatives to using curb in combination with guardrail include transitioning from a curbed section to an open shoulder section in advance of guardrail, or using a shorter open shoulder section with an impact attenuator instead of guardrail.

When no other alternative is feasible, curb may be used in combination with guardrail. See Chapter 5 of AASHTO's *Roadside Design Guide* for more specific guidance on the use of curbs in conjunction with traffic barriers.

Where upgrade or replacement of guardrail is necessary, existing curb/guardrail combinations should be evaluated on a case-by-case basis to determine the appropriate guardrail installation.

9-6.06 Placement on Slopes

Most roadside barriers are designed for and tested on level terrain. If the barrier is to be placed on slopes steeper than 10:1, a flexible (cable) barrier type should be used.

Rigid (concrete barrier) or semi-rigid barriers (guardrail) are typically placed on the outside edge of the shoulder and should not be placed on a slope that exceeds 10:1. Existing slopes steeper than 10:1 should be flattened where rigid or semi-rigid barriers are to be used.

Flexible barrier may be used on 6:1 or flatter slopes, with some limited use on 4:1 or flatter slopes; however, the lateral placement of cable barrier should be within the acceptable limits discussed in Section 9-5.02.04.

9-6.07 Guardrail - Terminal Treatments

9-6.07.01 Approaching End Treatment

In general, Department practice is to use a flared terminal unless site conditions render one infeasible (i.e., space is insufficient for the flare).

9-6.07.02 Trailing End Terminals

The following information applies to the terminal treatment for the trailing end of a roadside barrier:

1. <u>2-Way Roadways/2-Lane</u>

    a. If the trailing end is within the clear zone for the opposing direction of travel, the trailing end terminal should meet the same crashworthy criteria as the approaching end.

b. If the trailing end is outside of the clear zone, providing a crashworthy terminal on the trailing end is not necessary. The Type 1 Cable Anchorage system, as presented in the *Standard Drawings*, may be used at these locations.

2. 2-Way Roadways/Undivided Multilane – If the trailing end is outside of the clear zone for the opposing direction of travel, providing a crashworthy terminal on the trailing end is not necessary. The Type 1 cable anchorage system, as presented in the *Standard Drawings*, may be used at these locations, if preferred, but the use of no treatment (including guardrail) may also be considered.

3. 1-Way Roadways – Providing a crashworthy terminal on the trailing end is not necessary. The Type 1 Cable Anchorage system, as presented in the *Standard Drawings,* may be used at these locations, if deemed necessary.

9-6.08 Crossing Route or Driveway within Length of Need

A crossing route or driveway within the length of need may make it infeasible to use a standard guardrail installation. Typical design options include the following:

1. Relocating or Closing the Crossing Route/Entrance – This decision is the preferred solution and should be considered during project design. This decision should involve consideration of expected crash risk, barrier maintenance costs, project scope, cost and impacts to adjacent properties, and the environment. Relocating or closing the roadway or entrance is not always feasible, but typically provides the most positive solution to the roadside safety issue.

2. Terminating the Guardrail in Advance of the Crossing Route – When relocating or closing the roadway/entrance is not feasible and where the nominal length of need may fall within the intersecting roadway or just beyond it, the standard guardrail may be truncated with an approved terminal section or crash cushion in advance of the roadway. Flaring the guardrail away from the roadway can be combined with this concept to improve length of need coverage. The decision to address the need for guardrail in this application should be where judgment or analysis indicates guardrail is preferable (e.g., flat slopes, minimal drop off) to the additional hazard posed by a short radius guardrail installation.

3. Short-Radius Guardrail – This option should only be considered on low-speed roadways ($V \leq 45$ mph) and where no other alternatives are feasible. A short-radius guardrail system imposes constraints on how close it can be installed to a bridge or other obstruction to be shielded, what radius can be used, and how far it should run along the intersecting side road.

4. W-beam guardrail is the only current system that may be used for a short-radius installation. See Chapter 5 of the *Roadside Design Guide* for more specific guidance on the design of guardrail placed in a radius.

9-7.0 ROADWAY/BRIDGE RAIL INTERACTION

The Bridge Division is responsible for selecting the type of and design of the bridge rail. This section discusses the roadway design implications of bridge rails. These design elements should be coordinated with the Bridge Division.

9-7.01 Guardrail Transitions

9-7.01.01 Types

The *Standard Drawings* present guardrail-to-bridge-rail transitions that are acceptable for use on Department projects.

9-7.01.02 Length of Need

As with any roadside barrier, the guardrail-to-bridge-rail transition should be extended a sufficient distance upstream from the bridge rail to provide proper protection for out-of-control vehicles. The information in Table 9-6-A and Figure 9-6-A applies to this length of need. To simplify the application, the following information applies to bridge rail approach transitions:

- Table 9-7-A – This table applies to both sides of an approach transition on a divided roadway and to the right side of an approach on a 2-lane/2-way roadway.

- Table 9-7-B – This table applies to the left side of an approach on a 2-lane/2-way roadway.

- Equations –The equations in Table 9-6-A should be used to determine the length of need for cases where the clear zones, shoulder widths, or barrier flare rates differ from the assumed values in Table 9-7-A and Table 9-7-B.

The variables A, B, C, and D in the two tables are defined on the applicable sheets in the *Standard Drawings*.

Table 9-7-A
LENGTH OF NEED
(Bridge Rail Approaches)

Design Speed (mph)	Traffic Volume (ADT)	Flare Rate (a:b)	Assumed Clear Zone ($L_A = L_C$)	Runout Length (L_R) (ft)	Tangent Section (L_1)	Assumed Shoulder Width (L_2)	Typical End Terminal LON Contribution	"A"* (ft)	"B"* (ft)	No. of 12.5-ft Sections for "B"
70	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	360 330 290 250	18′ - 1¾″	10 ft 10 ft 10 ft 10 ft	25 ft	180.6458 168.1458 155.6458 143.1458	137.5 125.0 112.5 100.0	11 10 9 8
65	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	330 290 250 225	18′ - 1¾″	10 ft 10 ft 8 ft 8 ft	25 ft	168.1458 155.6458 155.6458 143.1458	125.0 112.5 112.5 100.0	10 9 9 8
60	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	300 250 210 200	18′ - 1¾″	8 ft 8 ft 8 ft 8 ft	25 ft	180.6458 155.6458 130.6458 130.6458	137.5 112.5 87.5 87.5	11 9 7 7
55	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	265 220 185 175	18′ - 1¾″	8 ft 8 ft 6 ft 5 ft	25 ft	155.6458 143.1458 130.6458 130.6458	112.5 100.0 87.5 87.5	9 8 7 7
50	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	230 190 160 150	18′ - 1¾″	8 ft 8 ft 6 ft 5 ft	25 ft	143.1458 130.6458 118.1458 118.1458	100.0 87.5 75.0 75.0	8 7 6 6
45	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	195 160 135 125	18′ - 1¾″	8 ft 8 ft 6 ft 4 ft	25 ft	130.6458 105.6458 105.6458 105.6458	87.5 62.5 62.5 62.5	7 5 5 5
40	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	160 130 110 100	18′ - 1¾″	8 ft 8 ft 6 ft 4 ft	25 ft	105.6458 93.1458 80.6458 80.6458	62.5 50.0 37.5 37.5	5 4 3 3
35	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	135 110 95 85	18′ - 1¾″	8 ft 8 ft 6 ft 4 ft	25 ft	93.1458 80.6458 80.6458 80.6458	50.0 37.5 37.5 37.5	4 3 3 3
30	Over 10,000 5000 - 10,000 1000 - 5000 Under 1000	30:1	30 ft	110 90 80 70	18′ - 1¾″	8 ft 8 ft 6 ft 4 ft	25 ft	80.6458 68.1458 68.1458 68.1458	37.5 25.0 25.0 25.0	3 2 2 2

** "A" and "B" in this table apply to both sides of a bridge approach on a divided highway and the right side of a bridge approach on a two-way facility. See Table 9-6-A for definition of terms. The Standard Drawings should be referenced for the design application.*

Table 9-7-B
LENGTH OF NEED
(Bridge Rail Approaches)

Design Speed (mph)	Traffic Volume (ADT)	Flare Rate (a:b)	Assumed Clear Zone ($L_A = L_C$)	Runout Length (L_R) (ft)	Tangent Section (L_1)	Assumed Lane Width(1)	Assumed Shoulder Width (L_2)	Typical End Terminal LON Contribution	"C"* (ft)	"D"* (ft)	No. of 12.5-ft Sections for "D"
70	Over 10,000	30:1	30 ft	360	18′ - 1¾″	12 ft	10 ft	25 ft	80.6458	37.5	3
	5000 - 10,000			330		12 ft	10 ft		80.6458	37.5	3
	1000 - 5000			290		12 ft	10 ft		68.1458	25.0	2
	Under 1000			250		12 ft	10 ft		68.1458	25.0	2
65	Over 10,000	30:1	30 ft	330	18′ - 1¾″	12 ft	10 ft	25 ft	80.6458	37.5	3
	5000 - 10,000			290		12 ft	10 ft		68.1458	25.0	2
	1000 - 5000			250		12 ft	8 ft		80.6458	37.5	3
	Under 1000			225		12 ft	8 ft		68.1458	25.0	2
60	Over 10,000	30:1	30 ft	300	18′ - 1¾″	12 ft	8 ft	25 ft	80.6458	37.5	3
	5000 - 10,000			250		12 ft	8 ft		80.6458	37.5	3
	1000 - 5000			210		12 ft	8 ft		68.1458	25.0	2
	Under 1000			200		12 ft	8 ft		68.1458	25.0	2
55	Over 10,000	30:1	30 ft	265	18′ - 1¾″	12 ft	8 ft	25 ft	80.6458	37.5	3
	5000 - 10,000			220		12 ft	8 ft		68.1458	25.0	2
	1000 - 5000			185		12 ft	6 ft		68.1458	25.0	2
	Under 1000			175		12 ft	5 ft		68.1458	25.0	2
50	Over 10,000	30:1	30 ft	230	18′ - 1¾″	12 ft	8 ft	25 ft	68.1458	25.0	2
	5000 - 10,000			190		12 ft	8 ft		55.6458	12.5	1
	1000 - 5000			160		12 ft	6 ft		68.1458	25.0	2
	Under 1000			150		11 ft	5 ft		68.1458	25.0	2
45	Over 10,000	30:1	30 ft	195	18′ - 1¾″	12 ft	8 ft	25 ft	68.1458	25.0	2
	5000 - 10,000			160		12 ft	8 ft		55.6458	12.5	1
	1000 - 5000			135		12 ft	6 ft		55.6458	12.5	1
	Under 1000			125		11 ft	4 ft		68.1458	25.0	2
40	Over 10,000	30:1	30 ft	160	18′ - 1¾″	12 ft	8 ft	25 ft	55.6458	12.5	1
	5000 - 10,000			130		12 ft	8 ft		43.1458	0.0	0
	1000 - 5000			110		12 ft	6 ft		43.1458	0.0	0
	Under 1000			100		11 ft	4 ft		43.1458	0.0	0
35	Over 10,000	30:1	30 ft	135	18′ - 1¾″	12 ft	8 ft	25 ft	43.1458	0.0	0
	5000 - 10,000			110		12 ft	8 ft		43.1458	0.0	0
	1000 - 5000			95		11 ft	6 ft		43.1458	0.0	0
	Under 1000			85		11 ft	4 ft		43.1458	0.0	0
30	Over 10,000	30:1	30 ft	110	18′ - 1¾″	12 ft	8 ft	25 ft	43.1458	0.0	0
	5000 - 10,000			90		12 ft	8 ft		43.1458	0.0	0
	1000 - 5000			80		11 ft	6 ft		43.1458	0.0	0
	Under 1000			70		11 ft	4 ft		43.1458	0.0	0

* *"C" and "D" in this table apply to the left side of a bridge approach on a two-lane, two-way facility.*

(1) *L_2 = Lane width + Shoulder width. See Table 9-6-A for definition of terms. The Standard Drawings should be referenced for the design application.*

9-7.02 Sidewalks/Shared Use Paths at Bridges

Proposed sidewalks or shared use paths approaching and/or crossing a bridge have several roadway and bridge rail design implications. Bridge rail/sidewalk/shared use path design details for low-speed and high-speed applications are presented below and in Figure 9-7-A.

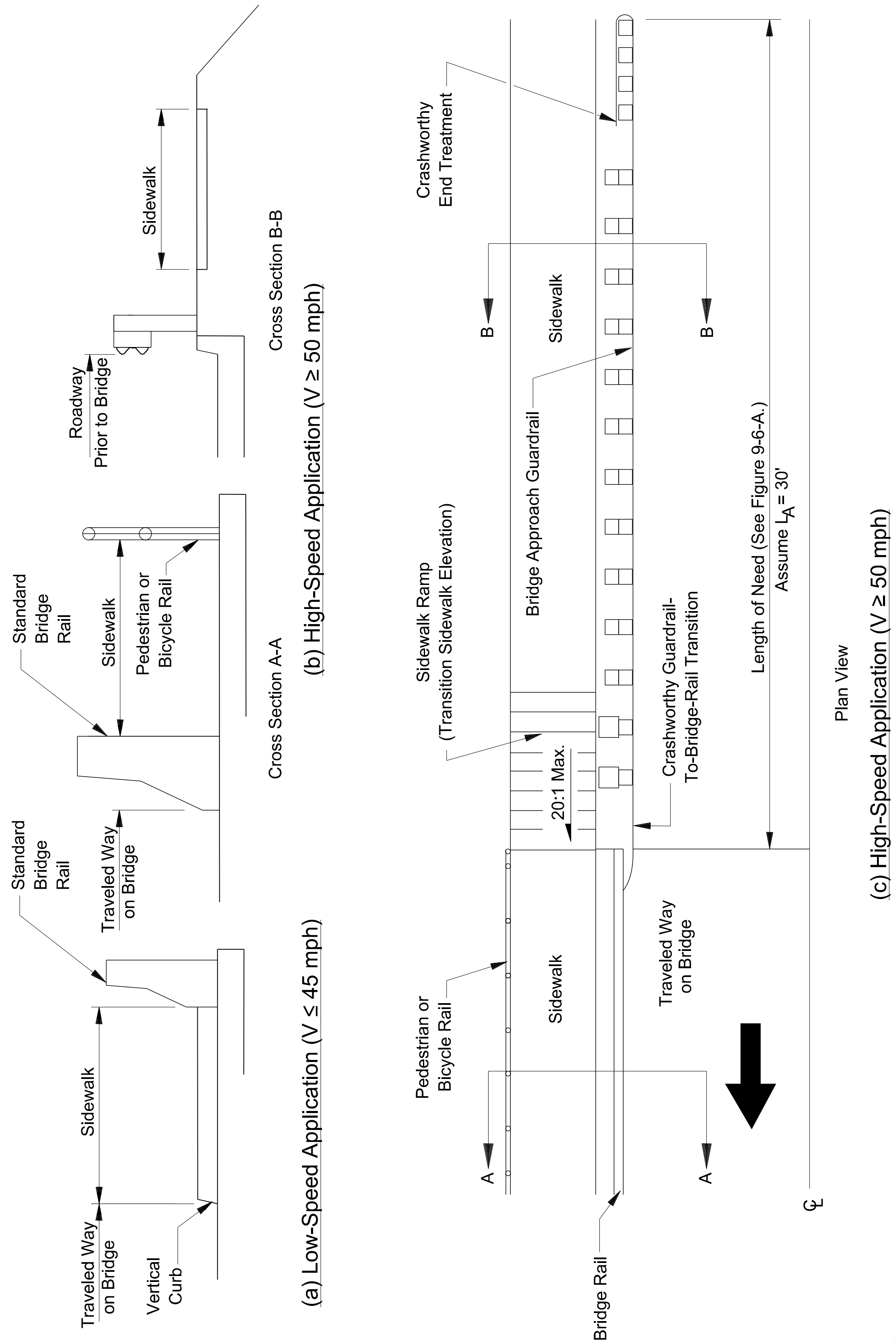
Where a sidewalk or shared use path is to be placed on a bridge, providing a bridge rail to separate vehicular traffic from pedestrians and/or bicyclists and a pedestrian or bicycle rail on the outside edge of the sidewalk or shared used path may be warranted. The following information should apply:

1. <u>High-speed roadways ($V \geq 50$ mph)</u> – The bridge rail should be placed between the traveled way and the sidewalk or shared use path. A pedestrian or bicycle rail should also be provided. The sidewalk or shared use path should not be raised (i.e., placed on the same elevation as the roadway surface).

2. <u>Low-speed roadways ($V \leq 45$ mph)</u> – Typically, the bridge rail should be located at the edge of the bridge deck or between the traveled way and the sidewalk or shared use path. A 42-inch concrete barrier bridge rail with the standard guardrail-to-bridge-rail transition should be used when the bridge rail is to be located at the edge of the bridge deck. Protection of pedestrians and/or bicyclists by use of a combination vehicular bridge rail/pedestrian/bicycle rail should be considered on a case-by-case basis. The following factors should be evaluated:

   a. design speed
   b. pedestrian/bicycle volumes
   c. vehicular traffic volumes
   d. crash pattern
   e. geometric impacts (e.g., sight distance)
   f. feasibility of providing proper end treatments
   g. construction costs

AASHTO's *LRFD Bridge Design Specifications* discusses the geometric configuration and structural loading for pedestrian and bicycle rails.



BRIDGE RAIL/SIDEWALK DESIGNS
Figure 9-7-A

9-8.0 IMPACT ATTENUATORS (CRASH CUSHIONS)

Impact attenuators are protective systems that prevent out-of-control vehicles from impacting hazards by decelerating the vehicle to a stop after a frontal impact or redirecting the vehicle away from the hazard after a side impact. Impact attenuators are adaptable to many roadside hazard locations where longitudinal barriers cannot feasibly be used. In addition, impact attenuators may be a preferred selection to barriers because of their smaller target area.

9-8.01 Warrants

Warrants for impact attenuators are the same as the barrier warrants discussed in section 9-3.0. Once a hazard is identified, preferred treatment options include removing the hazard, relocating the hazard, reducing the impact severity by using a breakaway support, or shielding the hazard with a barrier. If none of these options are feasible, then an impact attenuator should be considered. Impact attenuators are most often installed to shield fixed-point hazards. Examples include exit gore areas (particularly on structures), bridge piers, non-breakaway sign supports, and median barrier ends.

9-8.02 General

Impact attenuators operate on the principle of absorbing the energy of the vehicle through the use of bays or modules filled with crushable or plastically deformable materials. Some energy is also absorbed by the vehicle as the front end of the vehicle is crushed on impact. Impact attenuators typically include a rigid back-up support to contain the forces created by the deformation of the device. Most impact attenuators capture the vehicle in a frontal impact. For side impacts, the vehicle should be smoothly redirected by means of side panels and/or cables.

The manufacturer may be contacted for detailed information on specific impact attenuator installations.

9-8.02.01 Impact Speed

An initial impact speed should be selected for an impact attenuator. Table 9-8-A presents the Department’s criteria.

Table 9-8-A
IMPACT SPEED FOR IMPACT ATTENUATORS

Mainline Design Speed (mph)	Impact Speed (mph)	
	Freeways	Non-Freeways
$V \geq 60$	60	60
$45 < V < 60$	60	Design Speed
$V \leq 45$	—	45

9-8.02.02 Placement

Several factors should be considered in the placement of impact attenuators:

1. Level Terrain – The impact attenuator should be placed on a level surface or on a cross slope not to exceed 5%.

2. Curbs – Curbs should not be built at proposed new installations.

3. Surface – An asphalt or concrete pad should be provided under the impact attenuator.

4. Orientation – The proper orientation angle depends upon the design speed, roadway alignment, and lateral offset distance to the impact attenuator. For most roadside conditions, a maximum angle of approximately 10 degrees, as measured between the roadway and impact attenuator longitudinal centerlines, may be appropriate.

5. Location – The system should not infringe on the traveled way. A minimum of two feet should also be provided between the impact attenuator and the hazard to allow access to the system.

6. Bridge Joints – The placement of impact attenuators over bridge expansion joints or deflection joints in deep superstructures should be avoided because movement in these joints could create destructive strains on the system's anchor cables.

7. Transitions – Transitions between systems and backwalls, bridge rails, or other objects should be shaped to lessen the possibility of vehicular snagging. The yielding characteristics of the system should be considered when determining the transition.

9-9.0 REFERENCES

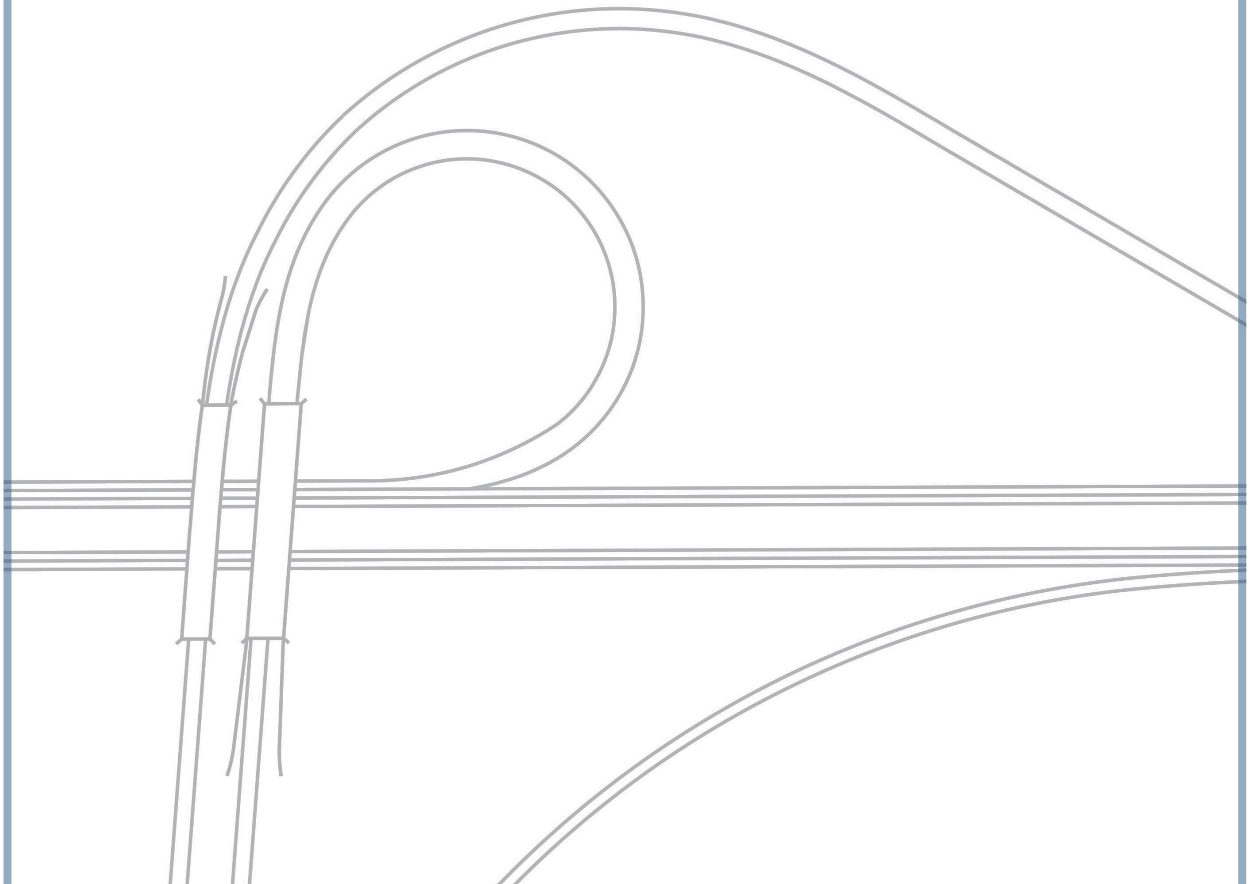
1. *Roadside Design Guide*, AASHTO, 2011.

2. "A Roadside Design Procedure," James Hatton, Federal Highway Administration, January 1974.

3. NCHRP 350 *Recommended Procedures for the Safety Performance Evaluation of Highway Features*, Transportation Research Board, 1993.

4. *Manual for Assessing Safety Hardware*, AASHTO, 2016.

5. *LRFD Bridge Design Specifications*, AASHTO, 2017.



CHAPTER 10

PERMANENT AND TEMPORARY TRAFFIC CONTROL

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Chapter 10
PERMANENT AND TEMPORARY TRAFFIC CONTROL

The *Manual of Uniform Traffic Control Devices* (*MUTCD*) contains criteria for traffic control devices used on state roadways. The following sections provide additional information based on the Department's criteria for permanent traffic control and temporary traffic control in work zones.

10-1.0 PERMANENT PAVEMENT MARKINGS

10-1.01 Criteria for No-Passing Zones

Pavement markings should be used to delineate no-passing zones on 2-lane, 2-way roadways. The Department has established 1000 feet of sight distance (for all design speeds) as its threshold for warranting pavement markings for no-passing zones. Where the available sight distance falls below 1000 feet along a horizontal or vertical curve, no-passing markings should be used.

A 3.5-foot eye height and object height should be used for measuring passing sight distances. The beginning of the no-passing zone should be the point at which the sight distance is less than 1000 feet. The end of the no-passing zone should be the point at which the sight distance exceeds 1000 feet. Solid yellow centerline markings should be used to establish no-passing zones where passing sight distance is unavailable or in other areas where passing should be prohibited.

Pavement marking details that indicate the limits of no-passing zones are typically not included in the Roadway Design Plans. Estimated striping quantities for passing and no-passing zones should be included in the plans; however, the proper locations for no-passing zones should be determined based on field observation and measurement during construction.

10-1.02 Pavement Marking

The *MDOT Pavement Marking Policy* should be referenced for permanent pavement markings on asphalt, concrete, and open-graded friction course pavement surfaces.

Contract Plans for roadway paving should include provisions for pavement markings, which may include painted stripes, thermoplastic pavement markings, plastic tape, rumble stripe, reflective markers, and pavement symbols. The following information applies to the presentation of pavement marking information in the Contract Plans:

1. Estimating Quantities – Pavement marking quantities should be estimated in terms of the pay item unit of measurement called for by the specifications as presented in Table 10-1-A.

2. Plan Sheets – Sufficient details for pavement markings should be shown in the plans by referencing the *Standard Drawings*, which may also be supplemented by detailed special design sheets for areas not addressed in the *Standard Drawings*.

Table 10-1-A
UNIT OF MEASURE FOR PAVEMENT MARKINGS

Type of Pavement Marking		Preferred Unit of Measure
Center Lines Lane Lines Pavement Edge Stripes Detail Traffic Stripes	(1) (1) (1) (2) (3)	Mile
No-Passing Lines Stop Bars	 (3)	Feet
Messages, Symbols, and Arrows		Square Feet
Raised Pavement Markers		Each

Notes:

1. *Separate estimates should be provided for each type of lane line (skip or continuous) and for each color (typically, white or yellow). The measurement of traffic stripes should be from end to end of each individual stripe. In the case of skip lines, the measurement should include nominal gap intervals and painted sections.*

2. *Detail traffic stripes should be measured from end to end of individual stripes. Quantities for plastic detail stripe are separated by color, but quantities for paint detail stripe are not.*

3. *Pay items should be based on 6-inch equivalent striping widths. Where a striping width greater than six inches is specified, the measured length should be adjusted by the ratio of the specified 6-inch equivalent width.*

10-2.0 PERMANENT SIGNING

10-2.01 General

Permanent signing is the primary mechanism for regulating, warning, and guiding traffic; therefore, all permanent signing should be in place before a roadway is opened to the travelling public.

The Roadway Design Division should coordinate with the Traffic Engineering Division in the design and preparation of the permanent signing plan sheets. Design characteristics of the roadway should determine the size and legend for a sign. As the design speed increases, larger signs may be warranted to provide adequate message comprehension time. The *MUTCD* and the *Standard Drawings* contain standard sign dimensions and specific legends for most signs. All signing should conform to the *MUTCD*.

The design procedure typically includes:

1. placing desired standard signs and guide signs at appropriate locations on the plan sheets and determining actual sizes of all guide signs
2. determining length and size of all sign supports

3. making detailed drawings of individual directional signs
4. identifying guidelines for delineators, object markers, and distance reference markers
5. computing quantities for all applicable pay items and preparing a summary of quantities
6. attending plan reviews and making recommended changes

Plan sheets usually will be developed using a scale of 1 inch = 200 feet; however, for interchanges and rest areas, a scale of 1 inch = 100 feet is preferred.

For statewide signing uniformity and continuity, providing signing beyond the project limits may be warranted.

10-2.02 Sign Placement

Placement criteria for roadway signs are documented in the *Standard Drawings* and in the *MUTCD*. Uniform placement of roadway signing, although desirable, is not always possible because roadway alignment and other factors may dictate the need for an alternative location. When determining sign locations, the following guidelines should be considered:

1. General – Signs are typically located on the right side of the roadway where they are readily recognized by road users. Other typical locations for signs are on overhead sign structures and in channelized islands. Signs in non-typical locations should be considered as supplemental to the signs in typical locations, except as otherwise provided in the *MUTCD*.

2. Dual Signing – Consideration should be given to dual signing (both sides of the roadway) where a critical warning, guidance, or regulatory sign message might be otherwise missed by the road user (due to heavy traffic on multilane roadways) or for additional conspicuity.

3. Geometric Design – Sign placement and geometric design should be coordinated as early as feasible during project planning and design.

4. Nighttime Visibility – Signs should be located to optimize nighttime visibility.

5. Longitudinal Placement – The *Standard Drawings* and the *MUTCD* provide information (e.g., proper sign spacing) for the longitudinal placement of signs based on the type of sign. Signs should be placed so as to allow road users adequate time to make a proper response in accordance with the *MUTCD*.

Sign locations should be adjusted to accommodate actual field conditions. In some cases, signs may be shifted longitudinally without compromising their intended purpose to optimize visibility and enhance safety. For example, the following locations should be avoided for sign placements:

a. just beyond the crest of a vertical curve
b. where a sign would be obscured by other signs, structures, or other roadway appurtenances
c. where a sign would obscure sight distance to approaching vehicles at intersections
d. where a sign would create an obstruction for pedestrians or bicyclists
e. where the visibility of a sign would be impaired by overhead illumination

6. Sign Groups – In general, signs are mounted individually on supports; however, a sign grouping (e.g., route markings) may be recommended instead. Consideration should be given to wind loading, driver ability to read and comprehend information, and breakaway criteria when designing sign groups.

7. Lateral Clearance – The *Standard Drawings* and the *MUTCD* contain minimum criteria for the lateral placement of signs. The following additional criteria should also apply:

   a. For signs adjacent to curbs, a 2-foot minimum offset should be provided between the face of the curb and the nearest edge of the sign.

   b. Large guide signs and motorist information signs should be located beyond the recommended clear zone where limited right of way or other physical constraints are not a factor.

   c. On steeper fill slopes, the possibility exists that an out-of-control vehicle could become partially airborne at the time of impact. Therefore, sign supports placed on fill slopes steeper than 6H:1V are only recommended where the face of the support is within two feet of the intersection of the shoulder slope and the foreslope.

8. Sign Posts/Supports – Crashworthy sign posts or breakaway sign supports should be used for signs located within the clear zone, and preferably, also for signs located beyond the clear zone. Breakaway features are not necessary on sign posts located behind traffic barriers. Longitudinal barriers should be installed to shield signs without crashworthy or breakaway features within the clear zone when no other options are available. On divided roadways, omni-directional break-away bases should be used where signs are subject to being struck from traffic in both directions (e.g., if there is an adjacent frontage road carrying 2-way traffic).

9. Regulatory and Warning Signs – Because these signs tend to lose their effectiveness when overused, regulatory and warning signs should be used conservatively.

10. Specific Service (Business Logo) Signs – Blue logo signs, which identify and display products or services of commercial business (e.g., food, gas, lodging), are not part of the permanent signing plans, but may still need to be considered when determining sign spacing.

10-2.03 Overhead Installation

Overhead signs should be used at locations where lane-use control is desired, and at locations where adequate space is not available along the roadside. The following conditions may be considered to determine if overhead signs could be beneficial:

1. traffic volume at or near capacity
2. complex interchange design
3. three or more lanes in a given direction
4. interchanges with lane drops and/or optional exit lanes
5. restricted sight distance
6. closely spaced interchanges

7. multilane exits
8. large percentage of trucks
9. high-speed traffic
10. consistency of sign message location throughout a series of interchanges
11. insufficient space for post-mounted signs
12. junction of two freeways

10-3.0 TRAFFIC SIGNALS

Traffic signals should be considered for installation at intersections where warranted by the *MUTCD*. The 20-year projection for geometric design of the roadway may be used for anticipating signal installations; however, a 5-year projection is typically used to determine if signals should be installed at non-signalized intersections.

If an investigation reveals the possible warrant for a signal installation, the Traffic Engineering Division may conduct an analysis of the location to determine if a signal should be installed.

Where an existing signal is located within a proposed construction project, the Traffic Engineering Division should be consulted to determine whether the existing signal should be replaced, adjusted to match the new construction, or removed. The Roadway Design Division should also coordinate with the Highway and Rail Safety Division for proposed traffic signals adjacent to at-grade railroad crossings.

The Traffic Engineering Division may provide assistance in developing the geometry of a signalized intersection. When a proposed signal is to be installed under a construction contract, the Traffic Engineering Division should be responsible for providing the traffic signal plans. The Roadway Design Division should coordinate with the Traffic Engineering Division throughout the plan development process to ensure that the signalized intersection is properly designed.

10-4.0 SCHOOL CROSSINGS

The Department should coordinate with local agencies to determine the appropriate application of signs and traffic signals for school crossings. TED Rule 941-7601-00-500 provides the Department's procedures on traffic control devices for school crossings.

10-5.0 TEMPORARY TRAFFIC CONTROL IN WORK ZONES

10-5.01 <u>General</u>

Roadway construction typically disrupts the normal traffic operations; therefore, traffic control in work zones should be considered during the design of each project. A traffic control plan (TCP) is a plan for the safe and effective movement of traffic during roadway construction and the safety of construction workers.

The Roadway Design Division typically provides a TCP for work zones while coordinating the development of the TCP with the Traffic Engineering Division, District, Construction Division, and other Divisions as necessary.

10-5.02 Traffic Control Plan

The TCP may range in scope from describing specific details of traffic accommodation to simply including the applicable *Standard Drawings*. The scope of the TCP should depend on the complexity and duration of the construction project. The *MUTCD* also provides additional information on the TCP.

The TCP may include, but is not limited to, the following:

1. temporary signing
2. temporary pavement markings
3. removal of existing pavement markings
4. queue protection
5. smart work zone
6. reduced construction speed limit
7. delineation and channelization
8. lane closures
9. temporary on-site detours
10. specific phase of construction details, including typical sections
11. placement of traffic control devices (e.g., temporary signals)
12. alternate pedestrian access routes for work zones
13. location and types of safety appurtenances (e.g., barriers, impact attenuators)
14. means of maintaining access to and from existing interchange ramps, intersections, roadways, and driveways
15. time that a specific traffic control feature (e.g., a flashing arrow board) should be in use (e.g., throughout construction period, only during lane closure)

10-5.03 Geometric Design

The following information should be applied to the geometric design of the TCP:

1. Design Speed – The design speed should typically be no more than 10 miles per hour below the posted speed limit for the approaching roadway. However, on low-volume roadways, a design speed that is more than 10 miles per hour below the approaching posted speed limit may be justified.

2. Sight Distance – Changes in geometric design of the approaching roadway often occur in work zones; therefore, available sight distance for the approaching driver should be considered. Lane closures, detours, or other transitions should desirably be located to provide Decision Sight Distance (DSD) for a vehicle approaching the work zone. See Table 4-5-B for minimum DSDs. At locations where it is not feasible to provide DSD in advance of the work zone, the sight distance that is provided should meet or exceed Stopping Sight Distance (SSD). When determining advance signing for a work zone, consideration should be given to whether additional signs should be included when DSD is not available for the work zone.

3. Lane/Shoulder Width – Engineering judgment should be used on a case-by-case basis to determine temporary lane or shoulder widths. However, temporary lane widths should be reduced no more than one foot from the recommended lane widths. Where portable

concrete barrier is used, the shoulder width between the adjacent travel lane and the barrier should desirably be a minimum of two feet, but should not be less than one foot.

4. Lane Closures – The taper rate for lane closures should conform to the following:

$$L = WS \qquad (S \geq 45 \text{ miles per hour}) \qquad \text{(Equation 10-5-1)}$$

$$L = \frac{WS^2}{60} \qquad (S \leq 40 \text{ miles per hour}) \qquad \text{(Equation 10-5-2)}$$

Where:

L = minimum length of transition, feet
W = width of offset, feet
S = posted speed limit before construction, miles per hour

5. Lane Shifts – For lane shifts, the minimum length of transition is 0.5 L, where L is as determined in Item 4 above. Other transition lengths (e.g., shoulder tapers, downstream tapers) are discussed in the *MUTCD*.

10-5.04 Roadside Safety

Positive protection devices should be provided on projects with long-duration work zones (two weeks or more) where workers have no means of escape (e.g., bridges), and on projects with pavement drop-offs greater than three inches that will remain in place overnight with less than eight feet of separation from traffic.

Consideration should be given to positive protection devices on projects with a design speed of 45 miles per hour or greater and on projects that place workers close to travel lanes open to traffic. The decision to provide positive protection should be made on a case-by-case basis. Additional factors that should be considered include, but are not limited to:

1. amount of traffic
2. duration of construction activity
3. nature of hazard (e.g., edge of traveled way drop-offs)
4. temporary 2-way traffic on one roadway of a divided facility (See Section 10-5.08.)
5. length of the work zone

The criteria provided in the *Standard Drawings* and in AASHTO's *Roadside Design Guide* should be followed for all work zone safety appurtenances. Roadside safety appurtenances in work zones should be crashworthy and may include the following, but are not limited to:

1. portable concrete median barrier
2. temporary guardrail
3. temporary work zone impact attenuators
4. portable water-filled barrier

10-5.05 Capacity Analysis

Reduced capacity through work zones may result in poor operation; therefore, work zone capacity should be estimated and compared to hourly traffic volumes. Lengthy backups and delays should desirably be avoided in the design of the traffic control plan. A capacity analysis may lead to significant modifications in the traffic control scheme. For example, lane closures may be prohibited during certain hours, or the sequence of construction may be altered to improve traffic operations.

The following are general guidelines for determining the capacity of interstates due to temporary lane closures:

- Short-Term Closures – The *Highway Capacity Manual* suggests a maximum volume of 1600 passenger cars per hour per lane assuming typical work activities. This maximum volume should be adjusted based on the intensity of the work activity, effects of heavy vehicles, presence of entrance ramps within 500 feet of the lane closure, and lane widths. The maximum capacity value could range from 1600 passenger cars per hour per lane for typical work activities to 1200–1400 passenger cars per hour per lane for unusual work activities, effects of heavy vehicles, and nearby entrance ramps.

- Long-Term Closures – Table 10-5-A provides suggested maximum capacity values for long-term lane closures. These values should be adjusted for reduced lane widths, experience, and engineering judgment.

Table 10-5-A
CAPACITY OF LONG-TERM LANE CLOSURES
(Passenger Cars/Hour/Lane)

Number of Normal Lanes to Reduced Lanes					
2 to 1	**3 to 2**	**3 to 1**	**4 to 3**	**4 to 2**	**4 to 1**
1400	1450	1450	1500	1450	1350

10-5.06 Detours

Detours may be either on-site (within the project site) or off-site (using existing roadways outside of the project site). For projects that utilize off-site detours, only state routes may be signed for the detour unless there is a prior written agreement with the local agency to use local roads.

The following criteria are recommended for typical on-site detours:

1. Design Speed – The design speed should typically be no more than 15 miles per hour below the posted speed limit of the approaching roadway, but should desirably be within 10 miles per hour of the posted design speed.

2. Minimum Radii – Table 10-5-B presents the minimum radii for temporary horizontal curves. These radii are based on the Department's horizontal alignment 3R criteria (Figure 12-2-A), assuming an adverse 2% cross slope is used.

3. Width – Table 10-5-C presents minimum width criteria for 2-lane, 2-way on-site detours.

4. Side Slopes – Slopes no steeper than 2:1 should be used for 2-lane, 2-way on-site detours. Slopes no steeper than 3:1 should desirably be used for on-site detours on multilane roadways with higher posted speeds (greater than 55 miles per hour).

5. Temporary Guardrail – The minimum length of temporary guardrail at bridge ends should be 37.5 feet. The *Standard Drawings* should be referenced for additional design details.

6. Advisory Speed Signs – For rural 2-lane roadways, signs should be included to reduce the advisory speed to 35 miles per hour in 10-mile per hour increments from the posted speed.

Table 10-5-B
MINIMUM RADII FOR TEMPORARY HORIZONTAL CURVES

Design Speed (mph)	R_{min} (ft)
30	400
35	600
40	800
45	1100
50	1400
55	1900
60	2600
65	3300
70	4500

Table 10-5-C
MINIMUM WIDTHS OF ON-SITE DETOURS
(2-Lane, 2-Way Roadways)

DHV	Roadway Width (ft)		Bridge Width (ft)
	Lanes	Shoulders	
≤ 200	12	3	24
> 200	12	5	24

10-5.07 Temporary Connections for Divided Roadways

Temporary connections may be provided to transition a divided roadway to a 2-way operation along one roadway, allowing construction work to proceed unimpeded by traffic. The TCP should be designed to discourage wrong-way traffic movements in such cases.

The design speed for a temporary connection should typically be no more than 10 miles per hour below the posted speed limit of the approaching roadway, but should desirably match the posted speed, especially if the temporary connection is expected to be in place for an extended period. Section 10-5.06 provides other geometric criteria for on-site detours.

Separation between the two traffic flows may be provided by drums, cones, precast concrete median barrier, or other appropriate devices. A project-specific analysis should be conducted to determine which type of separation is warranted. The analysis should consider traffic volumes, duration of separation, vehicular speeds, length of 2-way operation, geometrics (e.g., sight distance, lane widths, horizontal alignment), and construction costs.

10-5.08 Temporary Pavement Markings

Temporary pavement markings are typically provided for work zones to clearly delineate the intended path of travel for drivers. Paint is most often used for temporary striping; however, tape may be warranted in certain locations where the temporary stripe will not be removed or overlaid. Existing pavement markings in conflict with the proposed TCP should be removed. Temporary raised pavement markers should be provided whenever a work zone will be open to traffic for more than three months.

The *Standard Drawings* provide additional information on temporary pavement markings.

10-6.0 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2018

2. *Roadside Design Guide*, AASHTO, 2011

3. *Highway Capacity Manual 2010*, TRB, 2010

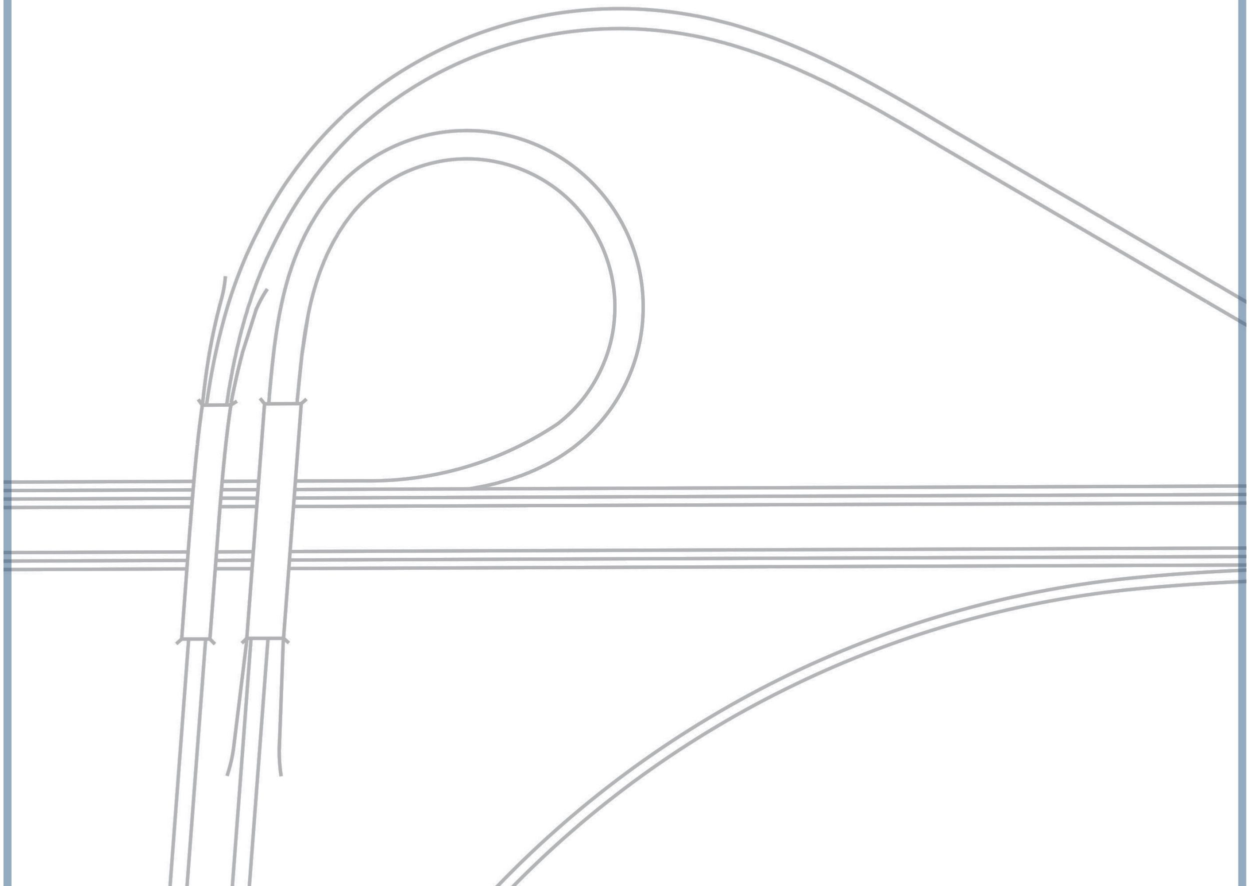
4. *Manual on Uniform Traffic Control Devices*, FHWA, ATSSA, AASHTO, ITE, 2009

5. *A Roadside Design Procedure*, James Hatton, Federal Highway Administration, January, 1974

6. NCHRP 350 *Recommended Procedures for the Safety Performance of Highway Features*, Transportation Research Board, 1993

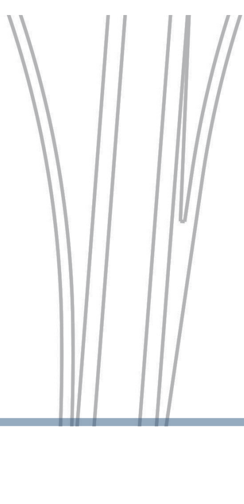
7. *Manual for Assessing Safety Hardware*, AASHTO, 2016

8. *Design Program, Saving Lives by Design*, Energy Absorption Systems, Inc.



CHAPTER 11

Right of Way and Fencing



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Chapter 11
RIGHT OF WAY AND FENCING

For projects where construction is not feasible within the existing right of way, proposed right of way should be acquired that provides sufficient room for construction and future maintenance of the roadway. However, proposed right of way would also minimize impacts to adjacent properties as much as feasible.

11-1.0 RIGHT OF WAY

11-1.01 <u>Right of Way Widths</u>

The right of way width is the sum of all cross-section elements including lane widths, shoulder widths, median width, side slopes, sidewalks, and border widths. The right of way border width is defined as the distance between the construction limits and the right of way lines, which provides room for construction activities during the project and maintenance activities after the project is complete. Construction limits represent the toe-of-fill slopes or top-of-cut slopes. Desirable right of way border widths are presented in the geometric design criteria tables in Chapters 2, 12, and 14.

In developed areas, limiting the right of way width may be desirable; however, the right of way width should not be less than that needed for all elements of the design cross section, utility accommodation, and minimum border widths.

The following steps describe how to determine the amount of right of way to be acquired for a project:

1. <u>Property Survey</u> – A property survey, which includes existing right of way and property lines, is typically included with the survey submitted to the Roadway Design Division. The property survey should be referenced on the plan-profile sheets.

2. <u>Preliminary Right of Way</u> – Preliminary Right of Way plans should include the proposed right of way that is expected to be the worst-case right of way for a project. Environmental Division uses Preliminary Right of Way plans to conduct studies to determine if any environmental impacts (e.g., impacts to historical properties, wetlands, businesses, residents) may need to be avoided, minimized, or mitigated. Preliminary Right of Way plans are also used by Right of Way Division to estimate the amount of parcels that are expected to be acquired.

   Preliminary Right of Way plans consist of conceptual horizontal and vertical alignments, but the roadway drainage design is not necessary to be incorporated into the plans until the Field Inspection Plans are being developed. However, temporary erosion control measures such as guide banks, silt basins, and clean water diversion ditches should be estimated to ensure that the preliminary right of way is the worst-case right of way for the project. Once the conceptual slope stakes for these design elements have been determined, the preliminary right of way is typically set with a minimum 50-foot border width.

While the preliminary right of way is only intended to be a worst-case scenario of the proposed right of way for the project, any residences, businesses, or any other right of way that would be considered unnecessary or too expensive to acquire to accomplish the needs of the project should desirably not be within the preliminary right of way.

3. Field Inspection – Field Inspection Plans should include the proposed right of way that is expected to be acquired for the project. All design elements that were estimated for Preliminary Right of Way Plans should be designed accordingly to ensure a sufficient amount of proposed right of way for construction of the project and future maintenance. The Field Inspection Team should review all of the design, including the amount of proposed right of way and all elements that affect the right of way, and make any recommended adjustments.

4. Draft Final Right of Way Plans – Draft Final Right of Way plans should include the recommended changes from the Field Inspection, and should be distributed to the Field Inspection Team for their review to ensure that all members are satisfied that the changes have been adequately addressed.

5. Survey, Maps, and Deeds (SMD) Plans – After the Draft Final Right of Way plans have been approved, the plans should be submitted to the SMD Section in the Roadway Design Division to prepare the acquisition maps and deeds for the project. The SMD Section may find minor adjustments that are needed at ties to existing right of way and/or property lines.

6. Final Right of Way – The Final Right of Way plans should include the corrections requested by the SMD Section as well as the completed acquisition maps and deeds.

7. Right of Way Revisions – After the Final Right of Way plans have been issued, any changes that affect the proposed right of way or access should result in a right of way revision. If a revision to the Final Right of Way plans is requested, the Roadway Design Section Engineer should request approval of the revision. The designer should then document that approval on the ROW Revision Sheet.

Once final right of way has been acquired or right of entry has been obtained for a parcel, a revision to the right of way cannot be made unless it is approved by the Assistant Chief Engineer of Preconstruction.

11-1.02 Setting the Proposed Right of Way

The following general criteria apply:

1. Right of way limits should be shown with as few breaks and changes in width as feasible; however, engineering judgment should be used to ensure an adequate buffer width is provided without acquiring an excessive amount of right of way.

2. Right of way marker locations should be labeled on the plan-profile sheet using the centerline station and offset from the right of way baseline. In most cases, the centerline of the mainline alignment should be used as the right of way baseline. Where local road reconstruction requires additional right of way, the proposed local road alignment should

be used as the right of way baseline. For interchange ramps, the ramp baseline should be used for identifying marker locations beyond the gore point of the ramp.

3. Desirably, the proposed right of way should parallel the roadway centerline. The radii of proposed right of way curves should be labeled so that they are easily distinguishable from proposed right of way lines. The proposed right of way should not be curved if it does not parallel the alignment.

4. Breaks in the proposed right of way should not occur in drainage channels, intersecting roadways, or driveways where installation of right of way markers would be infeasible.

11-1.03 Beginning or Ending of Project

When a project's beginning or ending right of way width falls within the limits of an individual property ownership and future connecting projects are expected, acquiring the remaining right of way width through the ownership should be considered to eliminate the need for further negotiations with the property owner.

11-1.04 Temporary Easements

A temporary easement may be obtained when it is not necessary to obtain permanent possession of the land. However, if post-construction maintenance is anticipated, permanent right of way should be acquired. Situations where a temporary easement may be acquired include, but are not limited to:

1. obtaining a proper grade or location for private driveways
2. temporarily draining ponds or lakes that conflict with construction
3. providing a feasible work area for proposed construction on urban projects (where a smaller buffer is used for the proposed right of way)
4. constructing and maintaining temporary silt basins for erosion control

11-1.05 Access Control

Access control is defined as the condition where the Department fully or partially controls the access to and from the roadway. The functional classification of a roadway (Section 2-1.01) is partially determined by the degree of access it allows. Access control may be exercised by statute, zoning, right of way purchases, driveway controls, turning and parking regulations, or geometric design (e.g., grade separations, frontage roads). Section 7-3.05 discusses access control at interchanges. Section 6-9.0 (rural driveways) and Section 14-2.07 (urban driveways) discuss procedures and criteria that may be used for driveway access control. Additional information on control of access is presented in the *Access Management Manual*.

Definitions for the basic types of access control are as follows:

1. Type 1 — Full Control (Freeway) – Full control of access is achieved by providing access only at interchanges with selected public roads. At-grade crossings or private driveway connections are not allowed.

2. Type 2 — Partial Control – Partial control of access is an intermediate level between full control and regulatory restriction. Priority is given to through traffic, but at-grade intersections and private driveway connections are allowed at specific locations. The two types of partial control of access are:

    a. Type 2A – Access to through traffic lanes is permitted only at designated exits and entrances (i.e., at-grade intersections). Frontage roads are typically provided for abutting property owners.

    b. Type 2B – Access to through traffic lanes is permitted only at designated exits and entrances (i.e., at-grade intersections, intermediate crossovers).

3. Type 3 — Control by Regulation (Conventional Roadway) – All roadways warrant some degree of access control. If access points are properly spaced and designed, the adverse effects on roadway capacity and safety can be minimized. On conventional roadways, vehicular ingress and egress from abutting properties directly to and from the through traffic lanes are permitted, except that direct access should be restricted as indicated in the *Access Management Manual* and Department Rules. The following Department Rules should apply:

    a. Rule No. 941-7501-03-001 “Processing of Permit Applications”

    b. Rule No. 941-7501-03-002 “Construction and Maintenance of Driveway, County Road, and Municipal Street Connections to State Highways”

    c. Rule No. 941-7501-04-013 “Driveway and Street Connections, Median Openings, Frontage Roads”

Controlled-access orders are required for projects that include new or revised Type 1 or Type 2 access control. Controlled-access orders are submitted by the Roadway Design Division Engineer for Commission approval. Once approved, the specific information for the controlled-access order should be included on the title sheet in an access control note (see Section 15-2.0).

11-1.06 Right of Way Markers

11-1.06.01 General Right of Way Marker Locations

Where feasible, proposed right of way markers should be located with stations and offsets at even 5-foot increments. Right of way markers should typically be located at PCs and PTs of horizontal curves (except for temporary roadways such as on-site detours), even though the stationing would generally not be at an even station. However, placing right of way markers where PCs or PTs would be located in drainage channels, intersecting roadways, or driveways is not feasible. Right of way markers at PCs and PTs allow curves to be more easily located in the field after construction.

11-1.06.02 Right of Way Markers at No-Access Returns

Right of way markers should be placed at no-access limit returns, except for:

- the 50-foot openings at intermediate crossovers on projects with Type 2 access

- no-access limits that do not follow right of way lines

See Figure 6-7-G for an illustration of typical right of way marker placement at a channelized intersection with no-access right of way.

11-1.06.03 Right of Way Markers on Property Lines or Existing Right of Way

Beginning with the Preliminary Right of Way Plans, right of way markers should be described with a station and an offset. When setting proposed right of way limits, a marker should preferably not be placed on a property line. However, if necessary to place a marker on a property line, the label should include the note “Tie to Property Line” in addition to the station and offset. Where the proposed right of way intersects the existing right of way, the marker label should include a similar note, “Tie to Exist. ROW”.

If a right of way marker does not tie to a property line or existing right of way as intended, the issue should be addressed before issuing the Final Right of Way Plans. If this determination is made after the Final Right of Way Plans have been issued, the correction should be made with a right of way revision.

11-1.06.04 Right of Way Marker Spacing

Generally, right of way markers should be spaced at no more than an approximate 1000-foot distance; however, site conditions may justify a slightly different spacing.

11-1.06.05 Right of Way Marker and Easement Coordinate Sheets

Right of way marker and easement coordinate sheets should be included as part of the Final Right of Way Plans and Contract Plans if the project includes the acquisition of right of way.

Because the station, offset, and X and Y values for right of way markers and temporary easement points are electronically placed on the coordinate sheets, the markers should be located exactly at the intended station and offset. Right of way marker and easement point coordinates should be recorded to the nearest hundredth (0.01) of a foot on the sheets. OpenRoads should be used to assign the station/offset of each marker. Right of way marker and easement coordinate sheets should also be created and include the station, offset, and X and Y values. Each right of way marker on the coordinate sheet should be numbered with the FMS Construction number, as well as an individual number starting with 100 that will be stamped onto the marker cap. Right of way markers should be grouped together based on the alignment to which they are tied and listed in ascending stationing order within each group. The mainline markers should be listed first, and markers along subsequent alignments should be listed next, starting with the first intersecting roadway.

11-2.0 FENCING

All interstate highways and other freeways with Type 1 access control should be fenced. In rare instances where access is improbable (e.g., areas with water, steep terrain), fencing may be omitted. Additional information can be found in AASHTO’s *An Informational Guide on Fencing Controlled Access Highways*.

Fencing should be located at or near the right of way line. Installations up to 10 feet inside the right of way may be considered permissible in rare cases where maintenance is a concern.

11-2.01 <u>Types</u>

Two basic types of fences used on freeways are the woven wire and the chain link fence. The District should be consulted in determining the individual fence characteristics (e.g., type of posts, height of chain link fence). Details of the types of fences, posts, gates, and examples of typical installations are provided in the *Standard Drawings*. General design criteria for fencing are discussed in the following sections.

11-2.01.01 Woven Wire (Rural Areas)

1. Timber posts should normally be used.
2. Concrete posts should be used within limits of interchange areas.
3. Timber posts should be used adjacent to rest areas to preserve natural appearance.
4. On multilane roadways with wide medians and where structures are installed for the passage of animals, a comparable fence should be used across the median on each side of the path to allow passage without permitting access to the roadway.

11-2.01.02 Chain Link (Urban or Developed Areas)

1. A 5-foot high chain link fence should normally be used.
2. A 6-foot high chain link fence should be used where pedestrian traffic is likely to be heavy.
3. At junkyard locations, a chain link fence should be used with a lattice height sufficient to screen the junkyard from the roadway.

11-2.02 <u>Estimating Quantities</u>

Estimates of the fencing quantities should be made according to the *Standard Specifications* and should include the following pay items:

1. length (in feet) of fence of the type and size to be installed, exclusive of openings
2. the number of posts by type of material, type of post (e.g., line post, brace post, gate post), and length
3. the number of gates of specified type and size
4. length (in feet) of single-strand barbed wire when not included in the standard installation

The method for estimating fencing quantities is as follows:

1. <u>Total Length of Fence</u> – The total length of fence (in feet) is a linear measurement, exclusive of openings, measured directly from the plan-profile sheets.

2. Total Number of Brace Posts – The total number of brace posts should be estimated as follows:

    a. Three brace posts at every fence turn
    b. One brace post at every fence gate
    c. Two brace posts at every 250 feet in curves
    d. Two brace posts at every 500 feet along tangents

3. Type of Brace Posts – To estimate the number of each brace post type, the following percent breakdown should be applied to the total number of brace posts determined in Item 2 above:

    a. 75% for 8-foot brace posts
    b. 15% for 10-foot brace posts
    c. 10% for 12-foot brace posts

4. Total Number of Line Posts – The total number of line posts should be determined as follows:

    Total Number of Line Posts = (Total Fence Length/12 feet) – Total Number of Brace Posts

5. Type of Line Posts – To estimate the number of each line post type, the following percent breakdown should be applied to the total number of line posts determined in Item 4 above:

    a. 75% for 7-foot line posts
    b. 15% for 9-foot line posts
    c. 10% for 10-foot line posts

6. Number of Steel Posts at Crossings – To determine the number of steel posts at each crossing (e.g., stream), the following criteria should be used:

    a. Type I crossings – no steel posts
    b. Type II crossings – three steel posts
    c. Type III crossings – two steel posts

    Examples of typical installations (Type I, II, and III crossings) are provided in the *Standard Drawings*.

7. Number of Gate Posts – One gate post for each gate.

8. Length of Barbed Wire at Crossings – To determine the length of barbed wire at crossings, the following criteria should be used:

    a. Type I crossings – 350 feet of barbed wire
    b. Type II crossings – 200 feet of barbed wire
    c. Type III crossings – 150 feet of barbed wire

Estimated fencing quantities should be provided on an estimated quantities sheet, with the total of all fencing items summarized on the summary of quantities sheet.

11-2.03 Public Involvement

If significant clearing is to be involved for a fencing project along an existing route, a public involvement effort (e.g., a public hearing) may be warranted. The expected amount of clearing that will be proposed for a fencing project should be included in the Environmental Document.

CHAPTER 12

Existing Roadways

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Chapter 12
EXISTING ROADWAYS

12-1.0 INTRODUCTION

This chapter presents the 3R criteria for rural and urban non-freeways on the State Highway System, 3R criteria for freeways, spot improvements on non-freeways, and 1R criteria for preventive maintenance projects on Federal-aid routes.

A majority of the Department's highway program involves work on existing roadways. The Department makes every effort to maximize the greatest overall benefit from the available funds. Therefore, the Department uses a different set of criteria, known as Resurfacing, Restoration, and Rehabilitation (3R), where it is not feasible to use the criteria for new construction/reconstruction projects. 3R design criteria allows for a more flexible and performance-based design. Using new construction/reconstruction criteria on an existing roadway that is performing satisfactorily typically leads to overdesign and excessive construction costs.

The use of 3R criteria for a project on an existing roadway does not imply that all geometric design elements within the project limits should meet 3R criteria. If any existing geometric elements within the project limits are not in compliance with 3R criteria but are documented as performing satisfactorily, upgrading those elements may not be needed. By limiting the scope of the project to focus only on the documented performance improvement needs, more funds will be available to be spent on other needs throughout the State Highway System. See Section 2-10.0 for procedures on Design Exceptions and Design Variances.

When the scope of a project does not increase the capacity of a roadway (e.g., adding a continuous two-way left turn lane (CTWLTL), adding an auxiliary lane), 3R criteria may be used rather than new construction/ reconstruction criteria.

3R criteria may be used on the outer limits of a new construction project where new construction/reconstruction criteria would not be feasible for transitioning to sections of the existing roadway.

3R criteria may also be used within the limits of a new construction project when it is not feasible to meet new construction/reconstruction criteria due to certain constraints, such as, but not limited to, right of way, environmental, constructability, or cost. See Section 2-10.0 for procedures on Design Exceptions and Design Variances.

The use of 1R criteria applies to pavement preventive maintenance projects on Federal-aid routes. Resurfacing, restoration, or rehabilitation projects with multiple lifts of asphalt should follow 3R criteria. See Section 12-6.0 for the various types of preventive maintenance projects.

12-2.0 RURAL NON-FREEWAYS (3R Criteria)

12-2.01 General

12-2.01.01 Objectives

From an overall perspective, 3R criteria are intended to allow a more cost-efficient use of the available funds for roadway projects. 3R criteria should apply to many aspects that determine a roadway's serviceability, including:

1. structural integrity of pavement, bridges, and culverts
2. drainage design to minimize ponding on the roadway, protect the pavement structure from failure, and prevent roadway flooding during the design-year storm
3. Level of Service (LOS)
4. adequacy of access to abutting properties
5. geometric design of the roadway
6. roadside safety
7. traffic control devices

12-2.01.02 Evaluation for Use of 3R Criteria

The following factors are typically involved in project evaluation:

Documentation of Existing Geometrics – The as-built plans are normally reviewed in conjunction with the Location Committee Meeting (LCM) to determine the existing geometrics within the project limits. This review should consider cross-section elements, horizontal and vertical alignment, bridge widths, intersection geometrics, hydraulics, and roadside safety design.

1. Safety Analysis – A safety analysis may be conducted if there is a crash pattern within the limits of the project. The Highway and Rail Safety Division should be consulted to obtain the crash data and evaluate the following:

    a. Crash Data Review versus Statewide Average (for that type of roadway) – This information provides an overall indication of locations within the project limits that have a crash pattern.

    b. Crash Analysis by Type – This information indicates if certain types of crashes are common within the project limits.

    c. Crash Analysis by Location – Crashes may cluster around certain locations (e.g., intersection, horizontal curve). If available, Safety Performance Functions (SPFs) should be used to determine the expected crash frequency of each location. The results of the SPF should be compared to the existing crash frequency.

2. Speed Studies – Existing speed studies near the project may be reviewed. In addition, a spot speed study at specific locations (e.g., in advance of a horizontal or vertical curve) may be desirable to assist in the determination of geometric design improvements.

3. Traffic Volumes – The current and predicted traffic volumes should be examined within the limits of the project. Section 12-2.02.02 discusses the criteria for roadway capacity analyses on existing roadways.

4. Right of Way Acquisition – 3R criteria typically allow for less acquisition of right of way than new construction/reconstruction criteria.

5. Pavement Condition – Projects may be programmed because of a significant deterioration of the pavement structure. The extent of deterioration should influence the decision on whether a project should be designed using the new construction/reconstruction, 3R, or 1R design criteria.

6. Geometric Design of Adjacent Roadway Sections – The geometric features and operating speeds of roadway sections adjacent to the project should typically be examined, which should include a consideration of factors such as driver expectancy, geometric design consistency, and proper transitions between sections of different geometric designs.

7. Physical Constraints – The physical constraints within the limits of a project on an existing roadway often determine what geometric improvements are feasible and cost-efficient. Examples of such constraints include topography, adjacent development, available right of way, utilities, and environmental constraints (e.g., wetlands, historical properties, culturally-sensitive areas).

12-2.02 Design Controls

12-2.02.01 Design Speed

The geometric design criteria tables in Section 12-2.09 present the 3R criteria for design speed for rural roadways. These speeds are typically lower than those for new construction/reconstruction projects. However, the design speed should be equal to or greater than the posted speed limit.

12-2.02.02 Roadway Capacity

The analytical techniques in the *Highway Capacity Manual* should be used to conduct a capacity analysis when deemed necessary. The following key factors could affect the results of a capacity analysis:

- Level of Service – The geometric design criteria tables present the 3R LOS threshold.

- Design Volume – All elements of the facility should meet the LOS threshold for a DHV and/or ADT determined for 10 years beyond the expected construction completion date.

The Planning Division should be contacted to obtain the traffic data used to determine the level of improvement. At a minimum, the current ADT, DHV, percent of trucks and buses, intersection traffic counts, and any known future traffic impact should be included.

12-2.03 Cross-Section Elements

12-2.03.01 Widths

The geometric design criteria tables in Section 12-2.09 present the 3R widths for various cross-section elements of rural roadways. In general, these widths have been established considering the typical constraints of existing roadways and are consistent with overall objectives of the Department's 3R criteria.

12-2.03.02 Safety Edges

Safety edges should be included on all roadway projects that include asphalt pavement with open shoulders, including projects on existing roadways that are designed with 3R criteria. Section 13-2.03.05 should be referenced to determine the appropriate safety edge detail(s).

12-2.03.03 Bridges

12-2.03.03.1 *Scope of Work*

Any number of bridges may be within the limits of a project on an existing roadway. The scope of work for the bridge(s) may include any of the following:

1. Bridge Replacement – Depending upon the extent of the structural deficiencies, replacing the entire bridge (i.e., superstructure and substructure) may be more economical. The Design Loading Structural Capacity should be HL-93.

2. Bridge Rehabilitation – If the existing superstructure (e.g. girders, deck) is in poor condition but the substructure is structurally sound, the superstructure may be rehabilitated or replaced as part of the project. If the superstructure is rehabilitated, the Design Loading Structural Capacity should be HS-20 for a bridge designed under the Standard Specifications or HL-93 for a bridge designed under the LRFD Specifications. If the superstructure is replaced, the Design Loading Structural Capacity should be HL-93, and the substructure (e.g., abutments, piers, footings, piles) should be analyzed to ensure that it meets the structural capacity of the LRFD Specifications.

   In addition, where the bridge deck is structurally sound but its width is inadequate, the superstructure may be rehabilitated solely to widen the bridge deck. In this case, the Design Loading Structural Capacity should be HS-20 for a bridge designed under the Standard Specifications and HL-93 for a bridge designed under the LRFD Specifications. Bridge deck widening may then need to include work on the substructure, and the substructure should be analyzed to ensure that it meets the structural capacity of the appropriate specifications.

3. Existing Bridge to Remain in Place – If an existing bridge is structurally sound and meets the design and state legal loading structural capacities, improving the geometrics of the bridge may not be cost efficient. The bridge is then considered to be an existing bridge to remain in place. See Section 12-2.03.03.2 for information on bridge width for existing bridges to remain in place.

Where only the bridge substructure warrants rehabilitative work, the bridge may be considered an existing bridge to remain in place.

4. Bridge Rails/Transitions – For rehabilitated bridges, the existing bridge rails and approaching guardrail-to-bridge-rail transitions may need to be upgraded to meet the current criteria. For existing bridges to remain in place within the limits of a project, the Bridge Division should evaluate the existing bridge rails to determine if they should be upgraded. The existing approaching bridge rail transitions should also be evaluated to determine if the guardrail-to-bridge-rail transitions should be upgraded to meet the current criteria. Chapter 9, “Roadside Safety”, provides information on guardrail-to-bridge-rail transitions.

12-2.03.03.2 *Bridge Width*

The following information should be applied in evaluating the width of bridges within the limits of a project on an existing roadway:

- Bridge Replacement – The scope of a project on an existing roadway may be to use 3R criteria for the roadway, but may also include total reconstruction of an existing bridge. In such cases, the minimum bridge width should equal the approach roadway width.

- Bridge Rehabilitation – For these scopes of work, the full approach 3R roadway width should be provided across the bridge.

- Existing Bridge to Remain in Place – The existing width of a bridge proposed to remain in place should be evaluated using the geometric design criteria tables. If the existing width does not meet these criteria, but there is no crash pattern associated with the width of the bridge, the width of the bridge should remain unchanged. However, consideration should be given to widening the bridge according to the geometric design criteria tables.

12-2.04 Horizontal Alignment

The horizontal alignment criteria presented in Chapter 3, “Horizontal Alignment”, and in the *Standard Drawings* also apply to projects using 3R criteria on existing roadways, except as discussed below.

In the absence of a crash pattern, an existing horizontal curve may remain in place without further evaluation. However, if a crash pattern exists due to the existing horizontal curve, or if the project scope necessitates improving the existing curve, the design criteria in Chapter 3, “Horizontal Alignment”, and in the *Standard Drawings* should be used to determine the proper combination of radius and superelevation to meet the appropriate 3R design speed.

12-2.05 Vertical Alignment

12-2.05.01 Grades

In the absence of a crash pattern, existing grades may remain in place without further evaluation. However, if a crash pattern exists due to the existing grades, or if the project scope necessitates regrading of the roadway, the roadway should be regraded to the extent feasible in accordance with the maximum grades listed in the new construction/reconstruction geometric design criteria tables in Chapter 2, "Basic Design Controls" or Chapter 14, "Geometric Design of Urban Roadways".

12-2.05.02 Crest Vertical Curves

In the absence of a crash pattern, an existing crest vertical curve may remain in place without further evaluation. However, if a crash pattern exists due to the existing curve, or if the project scope necessitates improving the existing curve, the design criteria in Chapter 4, "Vertical Alignment", should desirably be used to determine the proper length of vertical curve and K-value for the appropriate 3R design speed.

The K-values in Section 4-5.02 provide for SSD to a 2-foot height of object. However, at a minimum, the K-values in Table 12-2-A should be used for the design of the reconstructed curve. These K-values provide minimum SSD to a 3-foot height of object, which is approximately the height of the center brake light on a passenger car.

If an existing intersection is located adjacent to or within an existing crest vertical curve that does not provide the minimum Intersection Sight Distance (ISD), the curve should be considered for reconstruction to provide the minimum ISD. See Section 6-6.0 for more information.

A crest vertical curve may be omitted where the algebraic difference in grades is 0.5% or less.

Table 12-2-A
3R K-VALUES FOR CREST VERTICAL CURVES

Design Speed (mph)	SSD Rounded for Design (ft)	K-Values Rounded for Design $K = S^2/2596$
20	115	5
25	155	10
30	200	16
35	250	24
40	305	36
45	360	50
50	425	70
55	495	95
60	570	125
65	645	160
70	730	206

$$L = \frac{AS^2}{2596} = KA$$

Where:

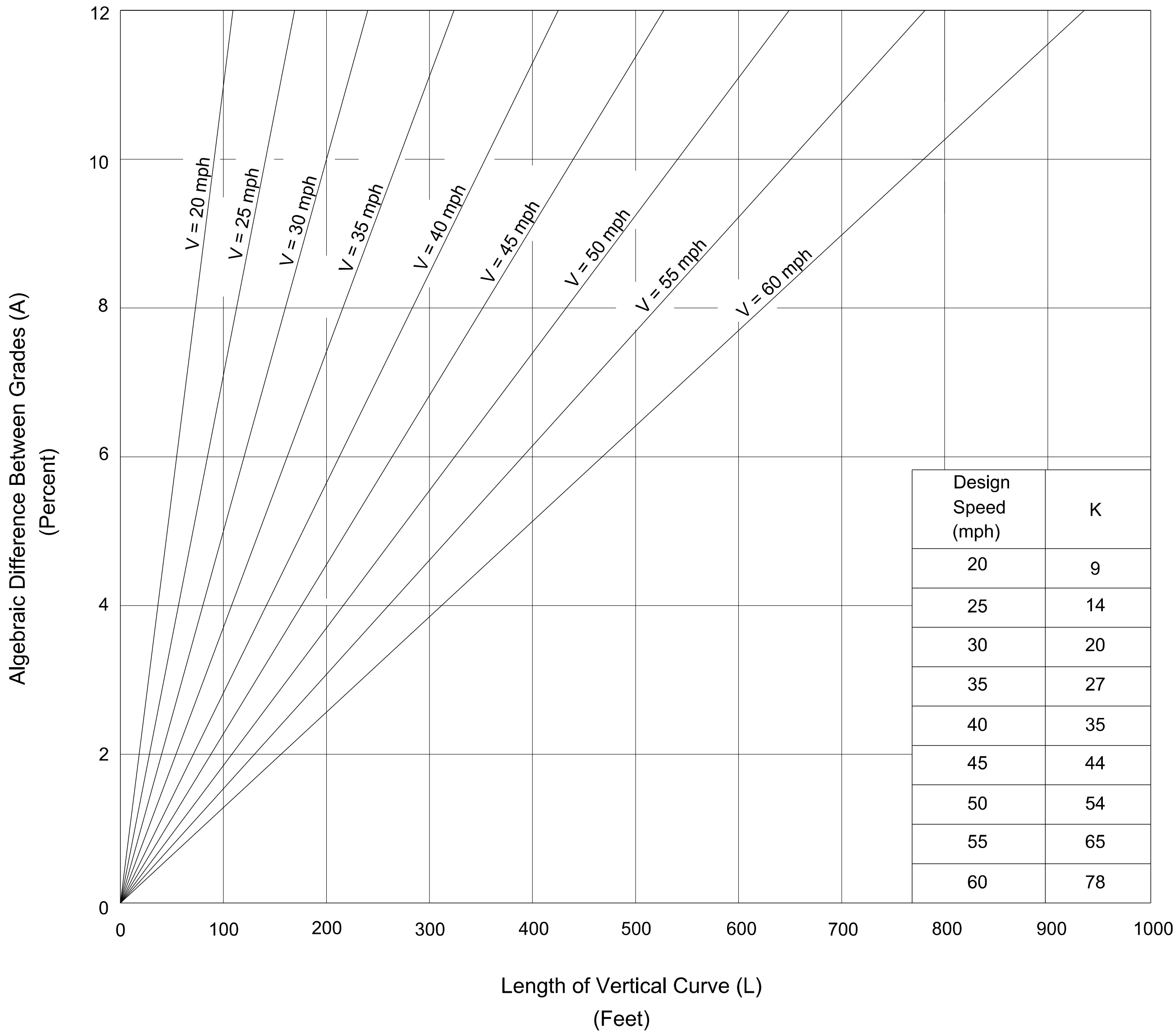
S = *sight distance for the design speed (feet)*
L = *length of vertical curve (feet)*
A = *algebraic difference between grades (%)*
K = *horizontal distance (in feet) to produce a 1% change in gradient*

12-2.05.03 Sag Vertical Curves

Similar to crest vertical curves, in the absence of a crash pattern, an existing sag vertical curve may remain in place without further evaluation. However, if a crash pattern exists due to the existing curve, or if the decision is made to improve the existing curve for other reasons, the design criteria in Chapter 4, “Vertical Alignment”, should desirably be used to determine the proper length of vertical curve and K-value for the appropriate 3R design speed.

The design of the reconstructed curve should desirably meet the criteria for headlight sight distance in Section 4-5.03, which allows the vehicle’s headlights to illuminate the pavement for a distance equal to the SSD for the design speed. If it is not feasible to meet the criteria for headlight sight distance, then the curve should meet or exceed the comfort criteria K values in Figure 12-2-A.

A sag vertical curve may be omitted where the algebraic difference in grades is 0.5% or less.



Design Speed (mph)	K
20	9
25	14
30	20
35	27
40	35
45	44
50	54
55	65
60	78

$$L = \frac{AV^2}{46.5}$$

$$L = KA$$

Where:

L	=	Length of vertical curve (feet)
A	=	Algebraic difference between grades (%)
K	=	Horizontal distance for a 1% change in gradient (feet)
V	=	Design speed (miles per hour)

3R K-VALUES FOR SAG VERTICAL CURVES
Figure 12-2-A

12-2.06 At-Grade Intersections

Chapter 6, "At-Grade Intersections", presents the Department's design criteria for the new construction/reconstruction of rural and urban at-grade intersections. Improvements to existing at-grade intersections on rural roadways should desirably meet or exceed this criteria. However, if a crash pattern related to the design of the existing intersection does not exist, the intersection may remain, or the improvements may be limited, using Chapter 6 as a reference, to only those improvements that are necessary in order to accomplish the goals of the project.

If the scope of a project requires that an intersection be reconstructed in its current location or at a new location, the reconstructed intersection should meet or exceed the criteria presented in Chapter 6.

12-2.07 Special Design Elements

Chapter 8, "Special Design Elements", provides the criteria and design details for many special design elements (e.g., erosion control, shared-use paths), which are also applicable to projects on existing roadways.

12-2.08 Roadside Safety

The roadside safety criteria in Chapter 9, "Roadside Safety", have been developed for new construction/reconstruction, which includes criteria for clear zones and barrier layout details (e.g., flare rates, length of need). The criteria in Chapter 9 also apply to 3R roadside safety design, but with the modifications as presented in the following subsections.

12-2.08.01 Clear Zones

Even when using 3R criteria, a clear zone that meets or exceeds the criteria in Section 9-2.0 should desirably be provided. Where this is not feasible, the recommended clear zone distances in Table 12-2-B should be used. The following recommendations should be followed:

- Safety Analysis – A safety analysis should be conducted to determine if there are any specific areas where crash patterns indicate that roadside safety improvements are warranted.

- Utilities – Utilities that are within the clear zone or are in conflict with proposed construction should be relocated.

- Safety Appurtenances – Installing barriers or impact attenuators is an alternative to providing a wider clear zone. Section 9-3.0 presents warrants for barriers. Barrier warrants are based on the relative severity between the hazard and barrier and do not address the question of whether or not a barrier installation is cost efficient. Therefore, for 3R, a decision should be made concerning whether or not a barrier should be installed to shield a hazard within the clear zone. Section 12-2.08.05 provides additional discussion.

- Bridge Piers – Bridges remaining in place that have piers within the clear zone should be protected. Roadway Design Division should coordinate with the Bridge Division to determine the appropriate treatments.

The clear zone distances in Table 12-2-B apply to recoverable fill slopes on rural roadways. For ditch sections, the following procedure should be used to determine the recommended clear zone distance:

1. Check Foreslope – Table 12-2-B should be used to determine the clear zone based on the foreslope.

2. Location of the Ditch – The bottom of the ditch should be checked to determine if it is within the clear zone. If the bottom of the ditch is beyond the clear zone, then only roadside hazards within the clear zone on the foreslope usually should be considered. For ditches within the clear zone, see Item 3 below.

3. Ditch Within the Clear Zone – The feasibility of modifying or relocating the ditch using one or more of the following methods should be considered:

    a. The cut foreslope may be flattened to reduce the recommended clear zone width. However, the designer should ensure that the ditch has an adequate ditch depth and slope to facilitate sufficient drainage.

    b. The cut foreslope may be lengthened to move the ditch outside of the clear zone. However, this approach would result in a deeper ditch that could cause undesirable right of way impacts.

    c. If it is infeasible to move the ditch outside of the clear zone, the feasibility of using a preferred channel cross section as discussed in AASHTO's *Roadside Design Guide* should be analyzed.

For auxiliary lanes (e.g., climbing lanes, passing lanes, turning lanes) 0.5 miles or less, the clear zone should be measured from the outside edge of the traveled way. For auxiliary lanes greater than 0.5 miles, the clear zone should be measured from the outside edge of the auxiliary lane.

12-2.08.02 Embankments (Fill Sections)

If feasible, any fill slope within the clear zone that is steeper than 3:1 should be flattened to 3:1 or flatter. If this is not feasible, installing guardrail should be considered where the height of the embankment is more than 10 feet.

Table 12-2-B
RECOMMENDED CLEAR ZONE DISTANCES (in feet) (3R Rural Non-Freeways)

Fill Slopes/Non-Curbed Facilities

Design Speed (mph)	Design ADT	Recoverable (Figure 9-2-A(a))		Non-Recoverable (Figure 9-2-A(b))
		6:1 or Flatter	5:1 to 4:1	
≤ 40	Under 750	3.5 – 5	3.5 – 5	SEE PROCEDURE IN SECTION 9-2.0.**
	750 – 1500	5 – 6	6 – 7	
	1501 – 6000	6 – 7	7 – 8	
	Over 6000	7 – 8	8 – 9	
45 – 50	Under 750	5 – 6	6 – 7	
	750 – 1500	7 – 8	8 – 10	
	1501 – 6000	8 – 9	10 – 13	
	Over 6000	10 – 11	12 – 14	
55	Under 750	6 – 7	7 – 9	
	750 – 1500	8 – 9	10 – 12	
	1501 – 6000	10 – 11	12 – 15	
	Over 6000	11 – 12	13 – 16*	
60	Under 750	8 – 9	10 – 12	
	750 – 1500	10 – 12	13 – 16*	
	1501 – 6000	13 – 15	16 – 20*	
	Over 6000	15 – 16*	18 – 22*	
65	Under 750	9 – 10	10 – 13	
	750 – 1500	12 – 13	14 – 18*	
	1501 – 6000	14 – 16*	17 – 21*	
	Over 6000	15 – 17*	19 – 23*	

Back Slopes

Design Speed (mph)	Design ADT	Recoverable (Figure 9-2-A(a))		Non-Recoverable (Figure 9-2-A(b))
		6:1 or Flatter	5:1 to 4:1	
≤ 40	Under 750	3.5 – 5	3.5 – 5	3.5 – 5
	750 – 1500	5 – 6	5 – 6	5 – 6
	1501 – 6000	6 – 7	6 – 7	6 – 7
	Over 6000	7 – 8	7 – 8	7 – 8
45 – 50	Under 750	4 – 5	4 – 5	5 – 6
	750 – 1500	5 – 6	5 – 7	7 – 8
	1501 – 6000	6 – 7	7 – 8	8 – 9
	Over 6000	7 – 8	9 – 10	10 – 11
55	Under 750	4 – 5	5 – 6	5 – 6
	750 – 1500	5 – 6	7 – 8	8 – 9
	1501 – 6000	7 – 8	8 – 9	10 – 11
	Over 6000	8 – 9	10 – 11	11 – 12
60	Under 750	5 – 6	6 – 7	7 – 8
	750 – 1500	6 – 7	8 – 9	10 – 11
	1501 – 6000	7 – 9	9 – 11	12 – 13
	Over 6000	10 – 11	12 – 13	13 – 14
65	Under 750	5 – 6	7 – 8	7 – 8
	750 – 1500	6 – 8	9 – 10	10 – 11
	1501 – 6000	8 – 10	11 – 12	13 – 14
	Over 6000	11 – 12	13 – 15	14 – 15

* *The clear zone distance may be limited to 15 feet for feasibility and to provide a consistent roadway template.*

** *The clear zone values from Table 12-2-B(a) should be adjusted using the procedure discussed in Section 9-2.02.*

Notes:

1. *All distances are measured from the edge of the traveled way except where long (greater than 0.5 miles) auxiliary lanes are present.*

2. *The Design ADT should be the total ADT for both directions of travel for the design year used for the project.*

12-2.08.03 Transverse Slopes

A roadway may intersect a driveway, side road, or median crossover, presenting a slope that could be impacted at an approximate 90-degree angle by out-of-control vehicles from the mainline. See Figure 9-3-A. On existing non-freeways, transverse slopes should meet the following criteria (either for retention of existing slopes or for flattening existing slopes):

- <u>Medians</u> – For existing medians on multilane roadways, the maximum transverse slope should be 10:1, which includes the ends of any drainage pipes that pass beneath a median crossover.

- <u>Facilities with Type 2 Access (Partial Control) (V ≥ 50 miles per hour)</u> – For transverse slopes other than in the median, the desirable slope is 10:1 and the maximum slope is 6:1, which includes the ends of any drainage pipes that pass beneath an intersecting roadway/driveway.

- <u>All Other Facilities</u> – In the absence of a crash pattern related to transverse slopes, existing transverse slopes may be retained. At certain locations, a drainage structure may be proposed beneath an intersecting roadway, driveway, or median crossover that is parallel to the traffic on the mainline. Wingwalls or flared ends for any parallel drainage structures within the clear zone should match the side slope of the intersecting roadway/driveway and/or include applicable safety treatments.

12-2.08.04 Roadside Ditches

In the absence of a crash pattern related to roadside ditches, all existing roadside ditch configurations may be retained.

12-2.08.05 Safety Appurtenances

During the design of a project on an existing roadway, all existing safety appurtenances should be evaluated to determine if they meet the latest safety performance and design criteria, including guardrail, median barriers, impact attenuators, sign supports, luminaire supports, etc. If the project will cause impacts to the existing safety appurtenances such that replacement is necessary, all safety appurtenances should follow the latest AASHTO guidance issued on *MASH* and related Department specifications and *Standard Drawings*. However, if the existing safety appurtenances are not impacted by the project, they may remain in place until such time that replacement is necessary.

The evaluation process for barrier warrants for projects on existing roadways is as follows:

1. The information provided in Sections 9-3.0 and 12-2.08.01 should be evaluated to determine if a barrier is recommended.

2. If an existing run of barrier is located where none is warranted, the barrier should preferably be removed.

3. For existing runs of barrier that require replacement due to the impacts of the project, the barrier should meet the applicable current performance and design criteria, including:

    a. operational acceptability (hardware, height, etc.)
    b. dynamic deflection criteria
    c. length of need
    d. flare rate
    e. lateral placement
    f. placement on slopes and behind curbs
    g. end treatments
    h. transitions

Barrier height should be considered on a pavement overlay or rehabilitation. Each existing barrier run should be analyzed individually. As a rule, consideration should be given to raising the barrier where its height after construction would not be within one inch of the recommended height for a new installation.

Lateral placement of roadside barriers should also be considered. Where guardrail is placed to shield a fill slope, two feet should be provided between the back of the barrier post and the break in the fill slope to ensure adequate soil support. On existing roadways, providing this minimum 2-foot distance may not be feasible, and placing the post at the break in the slope may be necessary instead. Where the fill slope is 3:1 or flatter, the performance of the barrier is not materially affected. However, where the slope is steeper than 3:1, a 7-foot post should be used if flattening the slope is not feasible.

12-2.09 3R Tables of Geometric Design Criteria

This section presents the Department's 3R geometric design criteria for existing rural non-freeway projects. Detailed information for alignment and cross-section elements can be found in Section 2-8.02 and 2-8.03, respectively, while 3R design criteria for each can be found in the following locations:

- Table 12-2-C and Table 12-2-D (rural arterials)
- Table 12-2-E and Table 12-2-F (rural collectors)
- Table 12-2-G (rural local roads)

Each table is followed by a set of footnotes that provides additional information for the items listed in the criteria tables.

When the design criteria proposed for projects on existing rural non-freeways do not comply with the values presented in the above-referenced tables, see Design Exception and Design Variance procedures presented in Section 2-10.0.

Table 12-2-C
GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS (2-LANE)
(3R Criteria)

	DESIGN ELEMENT			Manual Section	Design ADT			
					0 – 2000		Over 2000	
					T < 10%	T ≥ 10%	T < 10%	T ≥ 10%
Design Controls	Design Year			12-2.02	Desirable: 10 Years Minimum: Current			
Design Controls	*Design Speed **(2)**			12-2.02	≤ 55 mph			
Design Controls	Control of Access			11-1.05	Control by Regulation (Type 3)			
Design Controls	Level of Service Threshold			12-2.02	Desirable: B Minimum: D			
Cross-Section Elements	*Travel Lane Width			12-2.03	Des.: 12 ft Min.: 11 ft		12 ft	
Cross-Section Elements	Shoulder Width	*Usable		12-2.03	3 ft		5 ft	6 ft
Cross-Section Elements	Shoulder Width	Paved **(3)**		12-2.03	2 ft			
Cross-Section Elements	*Cross Slope	Travel Lane **(4)**		2-8.03 12-2.04	2%			
Cross-Section Elements	*Cross Slope	Shoulder		2-8.03 12-2.04	See Note **(5)**			
Cross-Section Elements	Auxiliary Lanes	Lane Width		2-8.03 12-2.06	Desirable: 12 ft Minimum: 11 ft			
Cross-Section Elements	Auxiliary Lanes	Usable Shoulder Width		2-8.03 12-2.06	3 ft			
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	*Design Loading Structural Capacity		12-2.03	See Note **(6)**			
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	Minimum Width **(6)**		12-2.03	Des.: 30 ft Min.: 28 ft		34 ft	36 ft
Cross-Section Elements	Existing Bridges to Remain in Place	*Design Loading Structural Capacity		12-2.03	See Note **(7)**			
Cross-Section Elements	Existing Bridges to Remain in Place	Minimum Width **(7)**		12-2.03	Traveled Way Width + 2 ft		Traveled Way Width + 4 ft	
Cross-Section Elements	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			2-8.03 11-1.01	15 ft - 20 ft			
Cross-Section Elements	Roadside Clear Zone	Guardrail		12-2.03 12-2.08	Usable Shoulder Width			
Cross-Section Elements	Roadside Clear Zone	Obstruction		12-2.03 12-2.08	See Note **(8)**			
Cross-Section Elements	Slope Schedule **(9)**	Cut	Foreslope (within clear zone)	2-8.03 12-2.08	4:1			
Cross-Section Elements	Slope Schedule **(9)**	Cut	Depth of Ditch	2-8.03 12-2.08	3 ft			
Cross-Section Elements	Slope Schedule **(9)**	Cut	Backslope	2-8.03 12-2.08	3:1			
Cross-Section Elements	Slope Schedule **(9)**	Fill	Safety Slope (within clear zone)	2-8.03 12-2.08	Desirable: 6:1 Maximum: 3:1			
Cross-Section Elements	Slope Schedule **(9)**	Fill	Fill Slope (outside clear zone)	2-8.03 12-2.08	Desirable: 3:1 Maximum: 2:1			
Alignment Elements	**DESIGN SPEED**				**45 mph**	**50 mph**	**55 mph**	
Alignment Elements	*Stopping Sight Distance			12-2.05	See Section 12-2.05			
Alignment Elements	Intersection Sight Distance **(10)**			6-6.0	500 ft	555 ft	610 ft	
Alignment Elements	*Superelevation Rate			2-8.02 12-2.04	See Section 12-2:04 e_{max} = 10%			
Alignment Elements	*Minimum Horizontal Curve Radius			2-8.02 12-2.04	See Section 12-2:04			
Alignment Elements	*Maximum Grades **(11)**	Level		2-8.02 12-2.05	Existing			
Alignment Elements	*Maximum Grades **(11)**	Rolling		2-8.02 12-2.05	Existing			
Alignment Elements	Minimum Grades			12-2.05	See Note **(12)**			
Alignment Elements	Vertical Curve (K-values)	*Crest		2-8.02 12-2.05	See Section 12-2.05			
Alignment Elements	Vertical Curve (K-values)	Sag		2-8.02 12-2.05	See Section 12-2.05			
Alignment Elements	*Vertical Clearance (arterial under) **(13)**	Reconstructed/ Rehabilitated Bridges		2-8.02	Desirable: 17 ft Minimum: 16 ft			
Alignment Elements	*Vertical Clearance (arterial under) **(13)**	Existing Bridges		2-8.02	Desirable: 16 ft Minimum: 14.5 ft			
Alignment Elements	*Vertical Clearance (arterial under) **(13)**	Sign Truss/ Pedestrian Bridge		2-8.02	19 ft			
Alignment Elements	Vertical Clearance (arterial over railroad) **(14)**			2-8.02	Desirable: 25 ft Minimum: 23.5 ft			

*For application of controlling design criteria, see Section 2-9.02.

Table 12-2-D
GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS (MULTILANE)
(3R Criteria)

	DESIGN ELEMENT			Manual Section	Multilane (1)
Design Controls	Design Year			12-2.02	Desirable: 10 Years Minimum: Current
	*Design Speed			12-2.02	65 mph
	Control of Access			11-1.05	Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)
	Level of Service Threshold			12-2.02	Desirable: B Minimum: D
Cross-Section Elements	*Travel Lane Width			12-2.03	12 ft
	Outside Shoulder Width		*Usable	12-2.03	4 ft
			Paved **(3)**		2 ft
	Median Shoulder Width		*Usable	12-2.03	4 ft
			Paved **(3)**		2 ft
	*Cross Slope		Travel Lane **(4)**	2-8.03 12-2.04	2%
			Shoulder		See Note **(5)**
	Auxiliary Lanes		Lane Width	2-8.03 12-2.06	Desirable: 12 ft Minimum: 11 ft
			Usable Shoulder Width		4 ft
	Median Width		Depressed	12-2.03	Existing
	Reconstructed/ Rehabilitated Bridges		*Design Loading Structural Capacity	12-2.03	See Note **(6)**
			Minimum Width		Approach Roadway Width
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	12-2.03	See Note **(7)**
			Minimum Width **(7)**		Traveled Way + 4 ft (each roadway)
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			2-8.03 11-1.01	15 ft - 20 ft
	Roadside Clear Zone		Guardrail	12-2.03 12-2.08	Usable Shoulder Width
			Obstruction		See Note **(8)**
	Slope Schedule **(9)**	Cut	Foreslope (within clear zone)	2-8.03 12-2.08	4:1
			Depth of Ditch		4 ft
			Backslope		3:1
		Fill	Safety Slope (within clear zone)		Desirable: 6:1 Maximum: 3:1
			Fill Slope (outside clear zone)		Desirable: 3:1 Maximum: 2:1
Alignment Elements	**DESIGN SPEED**				**65 mph**
	*Stopping Sight Distance			12-2.05	See Section 12-2.05
	Intersection Sight Distance			6-6.0	See Note **(10)**
	*Superelevation Rate			2-8.02 12-2.04	See Section 12-2.04 e_{max} = 10%
	* Minimum Horizontal Curve Radius			2-8.02 12-2.04	See Section 12-2.04
	*Maximum Grades **(11)**		Level	2-8.02 12-2.05	Existing
			Rolling		
	Minimum Grades			12-2.05	See Note **(12)**
	Vertical Curve (K-values)		*Crest	2-8.02 12-2.05	See Section 12-2.05
			Sag		
	*Vertical Clearance (arterial under) **(13)**		Reconstructed/ Rehabilitated Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft
			Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft
			Sign Truss/ Pedestrian Bridge		19 ft
	Vertical Clearance (arterial over railroad) **(14)**			2-8.02	Desirable: 25 ft Minimum: 23.5 ft

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 12-2-C and 12-2-D

1. *<u>Multilane Roadways</u> – The 3R criteria in the table for multilane roadways should apply to existing multilane roadways only. For an existing 2-lane roadway that will become a divided roadway, the new construction/reconstruction criteria provided in Table 2-9-C should be used.*

2. *<u>Design Speed</u> – A design speed as low as the posted speed limit may be used.*

3. *<u>Shoulder Surface Type (outside and median shoulders)</u> –*

    a. *Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved for special conditions.*

    b. *A minimum paved shoulder width of 10 feet, desirably 12 feet, should be provided for a distance of 350 feet beyond a reduction of the basic number of lanes (lane drop) of a multilane arterial for a recovery area for drivers. See Section 7-2.05 for more information.*

4. *<u>Cross Slope of Travel Lane (normal crown sections)</u> – The typical cross slope of the traveled way in a normal crown section is 2.0%. For arterials with three or more lanes sloped in the same direction, the cross slope may be increased to 2.5% for all lanes beyond the first two lanes that are sloped in the same direction. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

5. *<u>Cross Slope of Shoulder (normal crown sections)</u> –*

    a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

    b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

    c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

    d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

6. *<u>Reconstructed/Rehabilitated Bridges</u> –*

    a. *Design Loading Structural Capacity should equal HL-93 for reconstructed bridges. For Design Loading Structural Capacity on rehabilitated bridges, see Section 12-2.03.03.*

    b. *The widths shown in Table 12-2-C apply to a 2-lane arterial. The widths may be increased up to 44 feet if large equipment (i.e., farm equipment) is expected to cross the bridge. These widths should also be increased for auxiliary lanes.*

    c. *For bridges less than or equal to 200 feet in length, the bridge width should be no less than the full paved width of the approach roadway when the approach roadway width cannot be met.*

7. _Existing Bridges to Remain in Place_ –

    a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

    b. *Minimum widths provided in Tables 12-2-C and 12-2-D do not include auxiliary lanes or other existing features.*

    c. *The existing bridge width may remain unchanged if there is no crash pattern associated with the existing width. However, consideration should still be given to widening or reconstructing the bridge. If a crash pattern does exist, the bridge should be widened or reconstructed to the widths shown in the tables.*

8. _Roadside Clear Zone_ – *The recommended clear zones are based on design speed, side slopes, and traffic volumes. Section 12-2.08 provides additional information. All values are measured from the edge of the traveled way, except where long (greater than 0.5 miles) auxiliary lanes are present.*

9. _Slopes_ – *If high volume change soil (HVC) is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

10. _Intersection Sight Distance (ISD)_ – *The values provided in Table 12-2-C assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways. However, in the absence of a crash pattern, the existing ISD is typically acceptable without further evaluation. If the decision is made to improve the ISD, the improved design should desirably meet the criteria for new construction/reconstruction in Section 6-6.0. At existing intersections, removing obstructions within the sight triangle may not be feasible. In such cases, as much sight distance as feasible should be provided.*

11. _Maximum Grades_ – *The existing grades may remain unchanged if the project scope does not necessitate regrading of the roadway or if there is no crash pattern associated with the existing grades. However, if either condition exists, the roadway should be regraded to the extent feasible in accordance with the maximum grades listed in the new construction/reconstruction geometric design criteria tables in Chapter 2, "Basic Design Controls".*

12. _Minimum Grades_ –

    a. *On roadways, level gradients are acceptable on pavements that are adequately crowned to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    b. *On new or reconstructed bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

13. _Vertical Clearance (arterial under)_ –

    a. *The vertical clearance shown in the table should be provided over the entire arterial roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

c. For crossing routes going under the arterial roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.

d. Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).

14. <u>Vertical Clearance (arterial over railroad)</u> – If a project includes new or reconstructed bridges, the vertical clearance shown in the tables should typically be provided over the entire railroad right of way width.

Table 12-2-E
GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (2-LANE)
(3R Criteria)

	DESIGN ELEMENT	Design ADT / Trucks	Manual Section	0 – 1500 < 10%	0 – 1500 ≥ 10%	1501 – 2000 < 10%	1501 – 2000 ≥ 10%	Over 2000 < 10%	Over 2000 ≥ 10%
Design Controls	Design Year		12-2.02	Desirable: 10 Years Minimum: Current					
Design Controls	*Design Speed **(2)**		12-2.02	≤ 55 mph					
Design Controls	Control of Access		11-1.05	Control by Regulation (Type 3)					
Design Controls	Level of Service Threshold		12-2.02	Desirable: B Minimum: D					
Cross-Section Elements	*Travel Lane Width	V ≤ 45	12-2.03	11 ft		11 ft	11 ft	12 ft	
Cross-Section Elements	*Travel Lane Width	V ≥ 50	12-2.03	11 ft		11 ft	12 ft	12 ft	
Cross-Section Elements	Shoulder Width	*Usable	12-2.03	3 ft				5 ft	6 ft
Cross-Section Elements	Shoulder Width	Paved **(3)**	12-2.03	2 ft					
Cross-Section Elements	*Cross Slope	Travel Lane **(4)**	2-8.03 12-2.04	2%					
Cross-Section Elements	*Cross Slope	Shoulder	2-8.03 12-2.04	See Note **(5)**					
Cross-Section Elements	Auxiliary Lanes	Lane Width	2-8.03 12-2.06	Desirable: 12 ft Minimum: 11 ft					
Cross-Section Elements	Auxiliary Lanes	Usable Shoulder Width	2-8.03 12-2.06	3 ft					
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	*Design Loading Structural Capacity	12-2.03	See Note **(6)**					
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	Minimum Width **(6)** V ≤ 45	12-2.03	28 ft			28 ft	34 ft	36 ft
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	Minimum Width **(6)** V ≥ 50	12-2.03	28 ft			30 ft	34 ft	36 ft
Cross-Section Elements	Existing Bridges to Remain in Place	*Design Loading Structural Capacity	12-2.03	See Note **(7)**					
Cross-Section Elements	Existing Bridges to Remain in Place	Minimum Width	12-2.03	See Note **(7)**					
Cross-Section Elements	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)		2-8.03 11-1.01	15 ft – 20 ft					
Cross-Section Elements	Roadside Clear Zone	Guardrail	12-2.03 12-2.08	Usable Shoulder Width					
Cross-Section Elements	Roadside Clear Zone	Obstruction	12-2.03 12-2.08	See Note **(8)**					
Cross-Section Elements	Slope Schedule **(9)** – Cut	Foreslope (within clear zone)	2-8.03 12-2.08	V ≤ 45 mph: 3:1 V ≥ 50 mph: 4:1					
Cross-Section Elements	Slope Schedule **(9)** – Cut	Depth of Ditch	2-8.03 12-2.08	V ≤ 45 mph: 2 ft V ≥ 50 mph: 3 ft					
Cross-Section Elements	Slope Schedule **(9)** – Cut	Backslope	2-8.03 12-2.08	2:1					
Cross-Section Elements	Slope Schedule **(9)** – Fill	Safety Slope (within clear zone)	2-8.03 12-2.08	Desirable 6:1 Maximum 3:1					
Cross-Section Elements	Slope Schedule **(9)** – Fill	Fill Slope (outside clear zone)	2-8.03 12-2.08	Desirable 3:1 Maximum 2:1					
Alignment Elements	**DESIGN SPEED**			**45 mph**		**50 mph**		**55 mph**	
Alignment Elements	*Stopping Sight Distance		12-2.05	See Section 12-2.05					
Alignment Elements	Intersection Sight Distance **(10)**		6-6.0	500 ft		555 ft		610 ft	
Alignment Elements	*Superelevation Rate		2-8.02 12-2.04	See Section 12-2.04 e_{max} = 10%					
Alignment Elements	* Minimum Horizontal Curve Radius		2-8.02 12-2.04	See Section 12-2.04					
Alignment Elements	*Maximum Grades **(11)**	Level	2-8.02 12-2.05	Existing					
Alignment Elements	*Maximum Grades **(11)**	Rolling	2-8.02 12-2.05	Existing					
Alignment Elements	Minimum Grades		12-2.05	See Note **(12)**					
Alignment Elements	Vertical Curve (K-values)	*Crest	2-8.02 12-2.05	See Section 12-2.05					
Alignment Elements	Vertical Curve (K-values)	Sag	2-8.02 12-2.05	See Section 12-2.05					
Alignment Elements	* Vertical Clearance (collector under) **(13)**	Reconstructed/ Rehabilitated Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft					
Alignment Elements	* Vertical Clearance (collector under) **(13)**	Existing Bridges	2-8.02	Desirable: 16 ft Minimum: 14.5 ft					
Alignment Elements	* Vertical Clearance (collector under) **(13)**	Sign Truss/ Pedestrian Bridge	2-8.02	19 ft					
Alignment Elements	Vertical Clearance (collector over railroad) **(14)**		2-8.02	Desirable: 25 ft Minimum: 23.5 ft					

*For application of controlling design criteria, see Section 2-9.02.

Table 12-2-F
GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (MULTILANE)
(3R Criteria)

<table>
<tr><th></th><th colspan="3">DESIGN ELEMENT</th><th>Manual Section</th><th>Multilane (1)</th></tr>
<tr><td rowspan="4">Design Controls</td><td colspan="3">Design Year</td><td>12-2.02</td><td>Desirable: 10 Years Minimum: Current</td></tr>
<tr><td colspan="3">*Design Speed</td><td>12-2.02</td><td>65 mph</td></tr>
<tr><td colspan="3">Control of Access</td><td>11-1.05</td><td>Desirable: Partial (Type 2A or 2B)
Minimum: Control by Regulation (Type 3)</td></tr>
<tr><td colspan="3">Level of Service Threshold</td><td>12-2.02</td><td>Desirable: B Minimum: D</td></tr>
<tr><td rowspan="26">Cross-Section Elements</td><td colspan="3">*Travel Lane Width</td><td>12-2.03</td><td>12 ft</td></tr>
<tr><td colspan="2" rowspan="2">Outside Shoulder Width</td><td>*Usable</td><td rowspan="2">12-2.03</td><td>4 ft</td></tr>
<tr><td>Paved (3)</td><td>2 ft</td></tr>
<tr><td colspan="2" rowspan="2">Median Shoulder Width</td><td>*Usable</td><td rowspan="2">12-2.03</td><td>4 ft</td></tr>
<tr><td>Paved (3)</td><td>2 ft</td></tr>
<tr><td colspan="2" rowspan="2">*Cross Slope</td><td>Travel Lane (4)</td><td rowspan="2">2-8.03
12-2.04</td><td>2%</td></tr>
<tr><td>Shoulder</td><td>See Note (5)</td></tr>
<tr><td colspan="2" rowspan="2">Auxiliary Lanes</td><td>Lane Width</td><td rowspan="2">2-8.03
12-2.06</td><td>Desirable: 12 ft Minimum: 11 ft</td></tr>
<tr><td>Usable Shoulder Width</td><td>4 ft</td></tr>
<tr><td colspan="2">Median Width</td><td>Depressed</td><td>12-2.03</td><td>Existing</td></tr>
<tr><td colspan="2" rowspan="2">Reconstructed/ Rehabilitated Bridges</td><td>*Design Loading Structural Capacity</td><td rowspan="2">12-2.03</td><td>See Note (6)</td></tr>
<tr><td>Minimum Width (6)</td><td>Approach Roadway Width</td></tr>
<tr><td colspan="2" rowspan="2">Existing Bridges to Remain in Place</td><td>*Design Loading Structural Capacity</td><td rowspan="2">12-2.03</td><td rowspan="2">See Note (7)</td></tr>
<tr><td>Minimum Width</td></tr>
<tr><td colspan="3">Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)</td><td>2-8.03
11-1.01</td><td>15 ft - 20 ft</td></tr>
<tr><td colspan="2" rowspan="2">Roadside Clear Zone</td><td>Guardrail</td><td rowspan="2">12-2.03
12-2.08</td><td>Usable Shoulder Width</td></tr>
<tr><td>Obstruction</td><td>See Note (8)</td></tr>
<tr><td rowspan="5">Slope Schedule (9)</td><td rowspan="3">Cut</td><td>Foreslope (within clear zone)</td><td rowspan="5">2-8.03
12-2.08</td><td>4:1</td></tr>
<tr><td>Depth of Ditch</td><td>3 ft</td></tr>
<tr><td>Backslope</td><td>3:1</td></tr>
<tr><td rowspan="2">Fill</td><td>Safety Slope (within clear zone)</td><td>Desirable: 6:1 Maximum: 3:1</td></tr>
<tr><td>Fill Slope (outside clear zone)</td><td>Desirable 3:1 Maximum 2:1</td></tr>
<tr><td colspan="3">DESIGN SPEED</td><td></td><td>65 mph</td></tr>
<tr><td colspan="3">*Stopping Sight Distance</td><td>12-2.05</td><td>See Section 12-2.05</td></tr>
<tr><td colspan="3">Intersection Sight Distance</td><td>6-6.0</td><td>See Note (10)</td></tr>
<tr><td colspan="3">*Superelevation Rate</td><td>2-8.02
12-2.04</td><td>See Section 12-2.04 e_{max} = 10%</td></tr>
<tr><td rowspan="13">Alignment Elements</td><td colspan="3">*Minimum Horizontal Curve Radius</td><td>2-8.02
12-2.04</td><td>See Section 12-2.04</td></tr>
<tr><td colspan="2" rowspan="2">*Maximum Grades (11)</td><td>Level</td><td rowspan="2">2-8.02
12-2.05</td><td rowspan="2">Existing</td></tr>
<tr><td>Rolling</td></tr>
<tr><td colspan="3">Minimum Grades</td><td>12-2.05</td><td>See Note (12)</td></tr>
<tr><td colspan="2" rowspan="2">Vertical Curve (K-values)</td><td>*Crest</td><td rowspan="2">2-8.02
12-2.05</td><td rowspan="2">See Section 12-2.05</td></tr>
<tr><td>Sag</td></tr>
<tr><td colspan="2" rowspan="3">*Vertical Clearance (collector under) (13)</td><td>Reconstructed/ Rehabilitated Bridges</td><td rowspan="3">2-8.02</td><td>Desirable: 17 ft Minimum: 16 ft</td></tr>
<tr><td>Existing Bridges</td><td>Desirable: 16 ft Minimum: 14.5 ft</td></tr>
<tr><td>Sign Truss/Ped. Bridge</td><td>19 ft</td></tr>
<tr><td colspan="3">Vertical Clearance (arterial over railroad) (14)</td><td>2-8.02</td><td>Desirable: 25 ft Minimum: 23.5 ft</td></tr>
</table>

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 12-2-E and 12-2-F

1. *Multilane Roadways – The 3R criteria in the table for multilane roadways should apply to existing multilane roadways only. For an existing 2-lane roadway that will become a divided roadway, the new construction/reconstruction criteria provided in Table 2-9-E should be used.*

2. *Design Speed – A design speed as low as the posted speed limit may be used.*

3. *Shoulder Surface Type (outside and median shoulders) – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved for special conditions.*

4. *Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0%. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

5. *Cross Slope of Shoulder (normal crown sections) –*

   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

6. *Reconstructed/Rehabilitated Bridges –*

   a. *Design Loading Structural Capacity should equal HL-93 for reconstructed bridges. For Design Loading Structural Capacity on rehabilitated bridges, see Section 12-2.03.03.*

   b. *The widths shown in Table 12-2-E apply to a 2-lane collector. The widths may be increased up to 44 feet if large equipment (i.e., farm equipment) is expected to cross the bridge. These widths should also be increased for auxiliary lanes.*

   c. *For bridges less than or equal to 200 feet in length, the bridge width should be no less than the full paved width of the approach roadway when the approach roadway width cannot be met.*

7. *Existing Bridges to Remain in Place –*

   a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

   b. *The existing bridge width may remain unchanged if there is no crash pattern associated with the existing width. However, consideration should still be given to widening or reconstructing the bridge. If a crash pattern does exist, the bridge should be widened or reconstructed to the approach roadway width.*

8. <u>Roadside Clear Zone</u> – *The recommended clear zones are based on design speed, side slopes, and traffic volumes. Section 12-2.08 provides additional information. All values are measured from the edge of the traveled way, except where long (greater than 0.5 miles) auxiliary lanes are present.*

9. <u>Slopes</u> – *If high volume change (HVC) soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

10. <u>Intersection Sight Distance (ISD)</u> – *The values provided in Table 12-2-E assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways. However, in the absence of a crash pattern, the existing ISD is typically acceptable without further evaluation. If the decision is made to improve the ISD, the improved design should desirably meet the criteria for new construction/reconstruction in Section 6-6.0. At existing intersections, removing obstructions within the sight triangle may not be feasible. In such cases, as much sight distance as feasible should be provided.*

11. <u>Maximum Grades</u> – *The existing grades may remain unchanged if the project scope does not necessitate regrading of the roadway or if there is no crash pattern associated with the existing grades. However, if either condition exists, the roadway should be regraded to the extent feasible in accordance with the maximum grades listed in the new construction/reconstruction geometric design criteria tables in Chapter 2, "Basic Design Controls".*

12. <u>Minimum Grades</u> –

    a. *On roadways, level gradients are acceptable on pavements that are adequately crowned to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    b. *On new or reconstructed bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

13. <u>Vertical Clearance (collector under)</u> –

    a. *The vertical clearance shown in the table should be provided over the entire collector roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the collector roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

14. <u>Vertical Clearance (collector over railroad)</u> – *If a project includes new or reconstructed bridges, the vertical clearance shown in the tables should typically be provided over the entire railroad right of way width.*

Table 12-2-G
GEOMETRIC DESIGN CRITERIA FOR RURAL LOCAL ROADWAYS
(3R Criteria)

	DESIGN ELEMENT		Manual Section	Design ADT		
				Under 250	**250 – 1500**	**Over 1501**
Design Controls	*Design Speed **(1)**		12-2.02	30 mph	30 – 45 mph	See Table 12-2-E
	Control of Access		11-1.05	Control by Regulation (Type 3)		
	Level of Service Threshold		12-2.02	D		
Cross-Section Elements	*Travel Lane Width		12-2.03	10 ft	11 ft	
	Shoulder Width	*Usable	12-2.03	3 ft	3 ft	
		Paved **(2)**		2 ft	2 ft	
	*Cross Slope	Travel Lane **(3)**	2-8.03 12-2.04	2%		
		Shoulder		See Note **(4)**		
	Auxiliary Lanes	Lane Width	2-8.03 12-2.06	10 ft	11 ft	
		Usable Shoulder Width		3 ft	3 ft	
	Reconstructed/ Rehabilitated Bridges	*Design Loading Structural Capacity	12-2.03	See Note **(5)**		
		Minimum Width **(5)**		26 ft or Approach Roadway Width (whichever is greater)		
	Existing Bridges to Remain in Place	*Design Loading Structural Capacity	12-2.03	See Note **(6)**		
		Minimum Width				
	Roadside Clear Zone	Guardrail	12-2.03 12-2.08	Usable Shoulder Width		
		Obstruction		See Note **(7)**		
	Slope Schedule **(8)**	Cut Foreslope	2-8.03 12-2.08	3:1		
		Depth of Ditch		2 ft		
		Backslope		3:1		
		Fill Slope		3:1		

	DESIGN SPEED		Manual Section	30 mph	35 mph	40 mph	45 mph
Alignment Elements	*Stopping Sight Distance		12-2.05	See Section 12-2.05			
	Intersection Sight Distance **(9)**		6-6.0	335 ft	390 ft	445 ft	500 ft
	*Superelevation Rate		2-8.02 12-2.04	e_{max} = 6%			
	*Minimum Horizontal Curve Radius (e_{max} = 6%)		2-8.02 12-2.04	See Section 12-2.04			
	*Maximum Grades **(10)**	Level	2-8.02 12-2.05	Existing			
		Rolling					
	Minimum Grades		12-2.05	See Note **(11)**			
	Vertical Curve (K-values)	*Crest	2-8.02 12-2.05	See Section 12-2.05			
		Sag		See Section 12-2.05			
	*Vertical Clearance (local roadway under) **(12)**	Reconstructed/ Rehabilitated Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft			
		Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft			
	Vertical Clearance (local roadway over railroad) **(13)**		2-8.02	Desirable: 25 ft Minimum: 23.5 ft			

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Table 12-2-G

1. *<u>Design Speed</u> – A design speed as low as the posted speed limit may be used.*

2. *<u>Shoulder Surface Type</u> – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved for special conditions.*

3. *<u>Cross Slope of Travel Lane (normal crown sections)</u> – The typical cross slope of the traveled way in a normal crown section is 2.0%. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

4. *<u>Cross Slope of Shoulder (normal crown sections)</u> –*

   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

5. *<u>Reconstructed/Rehabilitated Bridges</u> –*

   a. *Design Loading Structural Capacity should equal HL-93 for reconstructed bridges. For Design Loading Structural Capacity on rehabilitated bridges, see Section 12-2.03.03.*

   b. *The width provided in the table applies to a 2-lane roadway. This width should be increased for additional lanes.*

6. *<u>Existing Bridges to Remain in Place</u> –*

   a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

   b. *The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side.*

   c. *The minimum width provided in Footnote 6b does not include auxiliary lanes or other existing features.*

   d. *The existing bridge width may remain unchanged if there is no crash pattern associated with the existing width. However, consideration should still be given to widening or reconstructing the bridge. If a crash pattern exists, the bridge should be widened or reconstructed to 26 feet or the approach roadway width, whichever is greater.*

7. *<u>Roadside Clear Zone</u> – The recommended clear zones are based on design speed, side slopes, and traffic volumes. Section 12-2.08 provides additional information. All values are measured from the edge of the traveled way, except where long (greater than 0.5 miles) auxiliary lanes are present.*

8. *Slopes – If high volume change (HVC) soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

9. *Intersection Sight Distance (ISD) – The values provided in Table 12-2-G assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways. However, in the absence of a crash pattern, the existing ISD is typically acceptable without further evaluation. If the decision is made to improve the ISD, the improved design should desirably meet the criteria for new construction/reconstruction in Section 6-6.0. At existing intersections, removing obstructions within the sight triangle may not be feasible. In such cases, as much sight distance as feasible should be provided.*

10. *Maximum Grades – The existing grades may remain unchanged if the project scope does not necessitate regrading of the roadway or if there is no crash pattern associated with the existing grades. However, if either condition exists, the roadway should be regraded to the extent feasible in accordance with the maximum grades listed in the new construction/reconstruction geometric design criteria tables in Chapter 2, "Basic Design Controls".*

11. *Minimum Grades –*

    a. *On roadways, level grades are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    b. *On new or reconstructed bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

12. *Vertical Clearance (local roadway under) –*

    a. *The vertical clearance shown in the table should be provided over the entire local roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the local roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

13. *Vertical Clearance (local roadway over railroad) – If a project includes new or reconstructed bridges, the vertical clearance shown in the tables should typically be provided over the entire railroad right of way width.*

12-3.0 URBAN NON-FREEWAYS (3R Criteria)

The criteria in this section apply to the use of 3R criteria for projects on existing urban roadways. Section 12-2.0 should be reviewed for additional information.

12-3.01 Evaluation for Use of 3R Criteria

Section 12-2.01 presents several factors that should be evaluated in the design projects on existing rural roadways. These evaluation factors also apply to the design of projects on existing urban roadways.

12-3.02 Design Controls

12-3.02.01 Design Speed

The geometric design criteria tables in Section 12-3.09 present the 3R criteria for design speed for urban roadways. These speeds are typically lower than those for new construction/reconstruction projects. The design speed should be equal to or greater than the posted speed limit.

12-3.02.02 Roadway Capacity

- Level of Service – The geometric design criteria tables present the 3R LOS threshold.

- Design Volume – All elements of the facility should meet the threshold for LOS for an ADT or DHV determined for 10 years beyond the expected construction completion date.

12-3.03 Cross-Section Elements

The geometric design tables referenced in Section 12-3.09 present the 3R criteria for cross-section elements for urban roadways. The information presented in Section 12-2.03 for rural roadways also applies to urban roadways. Section 14-2.06 should also be referenced for additional information related to urban roadways.

12-3.04 Horizontal Alignment

The horizontal alignment criteria presented in Chapter 14, “Geometric Design of Urban Roadways”, and in the *Standard Drawings* also apply to projects using 3R criteria on existing roadways, except as discussed below.

In the absence of a crash pattern, an existing horizontal curve may remain in place without further evaluation. However, if a crash pattern exists due to the existing horizontal curve, or if the decision is made to improve the existing curve for other reasons, the design criteria in Chapter 14, “Geometric Design of Urban Roadways”, and in the *Standard Drawings* should be used to determine the proper combination of radius and superelevation to meet the appropriate 3R design speed.

12-3.05 Vertical Alignment

Section 12-2.05 presents vertical alignment information for projects on existing rural roadways, which includes criteria for grades, crest vertical curves, and sag vertical curves. This information also applies to projects on existing urban roadways and is included in the geometric design criteria tables in Section 12-3.09.

12-3.06 At-Grade Intersections

Chapter 6, "At-Grade Intersections", presents the Department's design criteria for the new construction/reconstruction of rural and urban at-grade intersections. Improvements to existing at-grade intersections on urban roadways should desirably meet or exceed this criteria. However, if a crash pattern related to the design of the existing intersection does not exist, the intersection may remain or the improvements may be limited, using Chapter 6 as a reference, to only those improvements that are necessary in order to accomplish the goals of the project.

12-3.07 Sidewalks

Where projects on existing roadways include either proposed or existing pedestrian facilities, the amount of flexibility in design is dependent upon the scope of the project. If the scope consists of reconstructing the pedestrian facilities, or if the improvements to the existing roadway are such that the pedestrian facilities are impacted, the pedestrian facilities shall be replaced to the maximum extent feasible in accordance with the Department's accessibility criteria. These criteria meet or exceed the current Public Right-of-Way Accessibility Guidelines (*PROWAG*). See Section 14-2.06.05 for more information.

Where improvements to the existing roadway are such that the project results in no impacts to the pedestrian facilities, curb ramps are the only feature of the pedestrian facilities that shall be replaced to the maximum extent feasible in accordance with the Department's accessibility criteria. However, other improvements to the pedestrian facilities are encouraged to be made as part of the project.

Crosswalks constitute distinct elements of the right-of-way intended to facilitate pedestrian traffic. Regardless of whether there is curb-to-curb resurfacing of the roadway in general, resurfacing of a crosswalk also requires the provision of curb ramps at that crosswalk.

12-3.08 Roadside Safety

The information presented in Section 14-2.06.09 for new construction projects on urban roadways also applies to projects on 3R improvements to existing urban roadways.

12-3.09 3R Tables of Geometric Design Criteria

This section presents the Department's 3R geometric design criteria for existing rural non-freeway projects. Detailed information for alignment and cross-section elements can be found in Section 2-8.02 and 2-8.03, respectively, while 3R design criteria for each can be found in the following locations:

- Table 12-3-A and Table 12-3-B (urban arterials)
- Table 12-3-C and Table 12-3-D (urban collectors)

For 3R geometric design criteria for urban local roadways, see Table 14-2-J.

Each table is followed by a set of footnotes that provides additional information for the items listed in the criteria tables.

When the design criteria proposed for projects on existing urban non-freeways do not comply with the values presented in the above-referenced tables, see Design Exception and Design Variance procedures presented in Section 2-10.0.

Table 12-3-A
GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIALS (2-LANE/3-LANE)
(3R Criteria)

	DESIGN ELEMENT			Manual Section	With Curb	Without Curb
Design Controls	Design Year			12-3.02	Desirable: 10 years Minimum: Current	
	*Design Speed **(2)**			12-3.02	40 mph – 50 mph	45 mph – 55 mph
	Control of Access			11-1.05	Control by Regulation (Type 3)	
	Level of Service Threshold			14-2.01	Desirable: C Minimum: D	
Cross-Section Elements	*Travel Lane Width **(3)**			12-3.03	NHS: 12 ft Non-NHS: Des: 12 ft Min: 11 ft	
	Shoulder Width		*Usable **(4)**	12-3.03	Des: 8 ft Min: 2 ft	4 ft
			Paved		Same as Usable	See Note **(5)**
	*Cross Slope		Travel Lane **(6)**	2-8.03 12-3.04	2%	
			Shoulder		Shoulder Width > 4 ft: 4% Shoulder Width ≤ 4 ft: 2%	See Note **(7)**
	Auxiliary Lanes		Lane Width	2-8.03 12-3.06	Desirable: 12 ft Minimum: 11 ft	
			Usable Shoulder Width		Des: 4 ft Min: 2 ft	3 ft
	CTWLTL Width **(8)**			6-4.03	Desirable: 14 ft Minimum: 11 ft	
	Parking Lane Width **(9)**			14-2.06	Desirable: 12 ft Minimum: 8 ft	
	Sidewalk Width			14-2.06	Minimum: 5 ft	
	Reconstructed/ Rehabilitated Bridges		*Design Loading Structural Capacity	12-3.03	See Note **(10)**	
			Minimum Width		Approach Roadway Width	
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	12-3.03	See Note **(11)**	
			Minimum Width			
	Roadside Clear Zone		Guardrail	12-3.03 12-3.08	Usable Shoulder Width	
			Lateral Offset		1.5 ft **(12a)**	See Note **(12b)**
	Slope Schedule	Cut	Foreslope	12-3.08 14-2.06	+2%	4:1
			Depth of Ditch		N/A	3 ft
			Backslope		Des: 3:1 Max: 2:1	3:1
		Fill	Height ≤ 5 ft		Des: 3:1 Max: 2:1	Des: 4:1 Max: 3:1
			Height > 5 ft			3:1

	DESIGN SPEED		Manual Section	40 mph	45 mph	50 mph	55 mph
Alignment Elements	*Stopping Sight Distance		12-3.05	See Section 12-3.05			
	Intersection Sight Distance **(13)**		6-6.0	445 ft	500 ft	555 ft	610 ft
	*Superelevation Rate		2-8.02 12-3.04	See Section 12-3.04			
	*Minimum Horizontal Curve Radius		2-8.02 12-3.04	See Section 12-3.04			
	*Maximum Grades **(14)**	Level	2-8.02 12-3.05	Existing			
		Rolling					
	Minimum Grades		12-3.05	See Note **(15)**			
	Vertical Curve (K-values)	*Crest	2-8.02 12-3.05	See Section 12-3.05			
		Sag					
	*Vertical Clearance (arterial under) **(16)**	Reconstructed/ Rehabilitated Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft			
		Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft			
		Sign Truss/ Pedestrian Bridge		19 ft			
	Vertical Clearance (arterial over railroad) **(17)**		2-8.02	Desirable: 25 ft Minimum: 23.5 ft			

*For application of controlling design criteria, see Section 2-9.02.

Table 12-3-B
GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIALS (MULTILANE)
(3R Criteria)

	DESIGN ELEMENT			Manual Section	With Curb (1)	Without Curb (1)
Design Controls	Design Year			12-3.02	Desirable: 10 years	Minimum: Current
Design Controls	*Design Speed **(2)**			12-3.02	40 mph – 55 mph	45 mph – 60 mph
Design Controls	Control of Access			11-1.05	Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)	
Design Controls	Level of Service Threshold			14-2.01	Desirable: C	Minimum: D
Cross-Section Elements	*Travel Lane Width **(3)**			12-3.03	NHS: 12 ft Non-NHS: Des: 12 ft Min: 11 ft	
Cross-Section Elements	Outside Shoulder Width	*Usable **(4)**		12-3.03	Des: 10 ft Min: 2 ft	Des: 10 ft Min: 4 ft
Cross-Section Elements	Outside Shoulder Width	Paved		12-3.03	Same as Usable	Des: 8 ft Min: 3 ft
Cross-Section Elements	Median Shoulder Width	*Usable **(4)**		12-3.03	Des: 4 ft Min: 2 ft	Des: 8 ft Min: 4 ft
Cross-Section Elements	Median Shoulder Width	Paved		12-3.03	Same as Usable	Des: 3 ft Min: 3 ft
Cross-Section Elements	*Cross Slope	Travel Lane **(6)**		2-8.03 12-3.04	2%	
Cross-Section Elements	*Cross Slope	Shoulder		2-8.03 12-3.04	Shoulder Width > 4 ft: 4% Shoulder Width ≤ 4 ft: 2%	See Note **(7)**
Cross-Section Elements	Auxiliary Lanes	Lane Width		2-8.03 12-3.06	Desirable: Same as Travel Lane Min: 11 ft	
Cross-Section Elements	Auxiliary Lanes	Usable Shoulder Width		2-8.03 12-3.06	Des: 4 ft Min: 1 ft	4 ft
Cross-Section Elements	CTWLTL Width **(8)**			6-4.03	Desirable: 14 ft Minimum: 11 ft	
Cross-Section Elements	Parking Lane Width **(9)**			14-2.06	Desirable: 12 ft Minimum: 8 ft	
Cross-Section Elements	Sidewalk Width			14-2.06	Minimum: 5 ft	
Cross-Section Elements	Median Width			14-2.06	Existing	
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	*Design Loading Structural Capacity		12-3.03	See Note **(10)**	
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	Minimum Width		12-3.03	Approach Roadway Width	
Cross-Section Elements	Existing Bridges to Remain in Place	*Design Loading Structural Capacity		12-3.03	See Note **(11)**	
Cross-Section Elements	Existing Bridges to Remain in Place	Minimum Width		12-3.03	See Note **(11)**	
Cross-Section Elements	Roadside Clear Zone	Guardrail		12-3.03 12-3.08	Usable Shoulder Width	
Cross-Section Elements	Roadside Clear Zone	Lateral Clearance		12-3.03 12-3.08	1.5 ft **(12a)**	**(12b)**
Cross-Section Elements	Slope Schedule	Cut	Foreslope	12-3.08 14-2.06	+2%	4:1
Cross-Section Elements	Slope Schedule	Cut	Depth of Ditch	12-3.08 14-2.06	N/A	4 ft
Cross-Section Elements	Slope Schedule	Cut	Backslope	12-3.08 14-2.06	Des: 3:1 Max: 2:1	3:1
Cross-Section Elements	Slope Schedule	Fill	Height ≤ 5 ft	12-3.08 14-2.06	Des: 3:1 Max: 2:1	Des: 4:1 Max: 3:1
Cross-Section Elements	Slope Schedule	Fill	Height > 5 ft	12-3.08 14-2.06	Des: 3:1 Max: 2:1	3:1

	DESIGN SPEED		Manual Section	40 mph	45 mph	50 mph	55 mph	60 mph
Alignment Elements	*Stopping Sight Distance		12-3.05	See Section 12-3.05				
Alignment Elements	Intersection Sight Distance **(13)**		6-6.0	445 ft	500 ft	555 ft	610 ft	665 ft
Alignment Elements	*Superelevation Rate		2-8.02 12-3.04	See Section 12-3.04				
Alignment Elements	*Minimum Horizontal Curve Radius		2-8.02 12-3.04	See Section 12-3.04				
Alignment Elements	*Maximum Grades **(14)**	Level	2-8.02 12-3.05	Existing				
Alignment Elements	*Maximum Grades **(14)**	Rolling	2-8.02 12-3.05	Existing				
Alignment Elements	Minimum Grades		12-3.05	See Note **(15)**				
Alignment Elements	Vertical Curve (K-values)	*Crest	2-8.02 12-3.05	See Section 12-3.05				
Alignment Elements	Vertical Curve (K-values)	Sag	2-8.02 12-3.05	See Section 12-3.05				
Alignment Elements	*Vertical Clearance (arterial under) **(16)**	Reconstructed/ Rehabilitated Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft				
Alignment Elements	*Vertical Clearance (arterial under) **(16)**	Existing Bridges	2-8.02	Desirable: 16 ft Minimum: 14.5 ft				
Alignment Elements	*Vertical Clearance (arterial under) **(16)**	Sign Truss/ Pedestrian Bridge	2-8.02	19 ft				
Alignment Elements	Vertical Clearance (arterial over railroad) **(17)**		2-8.02	Desirable: 25 ft Minimum: 23.5 ft				

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 12-3-A and 12-3-B

1. *<u>Multilane Roadways</u> – The 3R criteria in the table for multilane roadways should apply to existing multilane roadways only. For an existing 2/3-lane roadway that will become a multilane roadway, the new construction/reconstruction criteria provided in Table 14-2-G should be used.*
2. *<u>Design Speed</u> – A design speed as low as the posted speed limit may be used.*
3. *<u>Travel Lane Width</u> –*
   a. *Existing lane widths on non-NHS routes may be retained in the absence of a crash pattern or if right of way and/or environmental constraints exist.*
   b. *11-foot lane widths may be used on NHS routes in more constrained areas and where truck and bus volumes are low. The lane widths needed for all lanes and intersection design controls should be evaluated collectively with consideration of all user modes and the adjacent land use.*
   c. *The lane width should desirably be increased by two feet for roadways that include angled on-street parking. See Section 14-2.06.06 for more information.*
4. *<u>Shoulder Width –</u>*
   a. *The minimum 2-foot shoulder width may be used on curbed roadways when right of way and/or other constraints exist. For any widths less than the desirable width, the drainage design should include consideration of the shoulder width in order to minimize ponding on the roadway.*
   b. *See Section 14-2.06.01 for information on shoulder width relative to curb and gutter usage.*
5. *<u>Shoulder Surface Type (outside and median shoulders)</u> – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved for special conditions.*
6. *<u>Cross Slope of Travel Lane (normal crown sections)</u> – The typical cross slope of the traveled way in a normal crown section is 2.0%. For arterials with three or more lanes sloped in the same direction, the cross slope may be increased to 2.5% for all lanes beyond the first two lanes that are sloped in the same direction. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*
7. *<u>Cross Slope of Shoulder (normal crown sections)</u> –*
   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*
   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*
   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*
   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*
8. *<u>CTWLTL Width</u> – In industrial areas with heavy truck traffic, the CTWLTL width should desirably be 16 feet. Section 6-4.03 provides more information on the CTWLTL width in areas of restricted right of way.*
9. *<u>Parking Lanes</u> – The widths shown in the tables only apply to parallel parking. Section 14-2.06.06 provides information on the design of angled parking.*

10. *<u>Reconstructed/Rehabilitated Bridges</u> – Design Loading Structural Capacity should equal HL-93 for reconstructed bridges. For Design Loading Structural Capacity on rehabilitated bridges, see Section 12-2.03.03.*

11. *<u>Existing Bridges to Remain in Place</u> –*

    a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

    b. *Undivided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side.*

    c. *Divided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders each side for each roadway.*

    d. *The minimum widths provided in footnotes 11b and 11c do not include other existing features, such as turn lanes, bicycle lanes, or sidewalks.*

    e. *The existing bridge width may remain unchanged if there is no crash pattern associated with the existing width. However, consideration should still be given to widening or reconstructing the bridge. If a crash pattern does exist, the bridge should be widened or reconstructed to the widths shown in footnotes 11b, 11c, and 11d.*

    f. *See Section 14-2.06.05 for information on sidewalks at existing bridges.*

12. *<u>Roadside Clear Zones</u>*

    a. *<u>With Curb</u> – Desirably, the clear zone for open shoulder rural sections (Section 9-2.0) should also be provided for curbed sections. The 1.5-foot lateral offset should be measured from the face of the curb to the obstruction, regardless of shoulder width.*

    b. *<u>Without Curb</u> – The clear zone for open shoulder rural sections (Section 9-2.0) should be provided. Where the recommended clear zone distances cannot be met, a 12-foot lateral offset (measured from edge of traveled way) should be provided on the outside of horizontal curves, and an 8-foot lateral offset should be provided elsewhere.*

13. *<u>Intersection Sight Distance (ISD)</u> – The values provided in Table 12-3-A assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways. However, in the absence of a crash pattern, the existing ISD is typically acceptable without further evaluation. If the decision is made to improve the ISD, the improved design should desirably meet the criteria for new construction/reconstruction in Section 6-6.0. At existing intersections, it may not be feasible to remove obstructions within the sight triangle. In such cases, the designer should provide as much sight distance as feasible.*

14. *<u>Maximum Grades</u> – The existing grades may remain unchanged if the project scope does not necessitate regrading of the roadway or if there is no crash pattern associated with the existing grades. However, if either condition exists, the roadway should be regraded to the extent feasible in accordance with the maximum grades listed in the new construction/reconstruction geometric design criteria tables in Chapter 14, "Basic Design Controls".*

15. *Minimum Grades* –

    a. *On roadways, level gradients are acceptable on pavements that are adequately crowned to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    b. *On new or reconstructed bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

16. *Vertical Clearance (arterial under)* –

    a. *The vertical clearance shown in the tables should be provided over the entire arterial roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the arterial roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

17. *Vertical Clearance (arterial over railroad) – If a project includes new or reconstructed bridges, the vertical clearance shown in the tables should typically be provided over the entire railroad right of way width.*

Table 12-3-C
GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTORS (2-LANE/3-LANE)
(3R Criteria)

	DESIGN ELEMENT		Manual Section	With Curb	Without Curb
Design Controls	Design Year		12-3.02	Desirable: 10 years Minimum: Current	
Design Controls	*Design Speed **(2)**		12-3.02	30 mph – 50 mph	
Design Controls	Control of Access		11-1.05	Control by Regulation (Type 3)	
Design Controls	Level of Service Threshold		14-2.01	Desirable: C Minimum: D	
Cross-Section Elements	*Travel Lane Width **(3)**		12-3.03	NHS: 12 ft Non-NHS: Des: 12 ft Min: 11 ft	
Cross-Section Elements	Outside Shoulder Width	*Usable **(4)**	12-3.03	Des: 8 ft Min: 2 ft	3 ft
Cross-Section Elements	Outside Shoulder Width	Paved	12-3.03	Same as Usable	See Note **(5)**
Cross-Section Elements	*Cross Slope	Travel Lane **(6)**	2-8.03 12-3.04	2%	
Cross-Section Elements	*Cross Slope	Shoulder	2-8.03 12-3.04	Shoulder Width > 4 ft: 4% Shoulder Width ≤ 4 ft: 2%	See Note **(7)**
Cross-Section Elements	Auxiliary Lanes	Lane Width	2-8.03 12-3.06	Desirable: Same as Travel Lane Min: 10 ft	
Cross-Section Elements	Auxiliary Lanes	Usable Shoulder Width	2-8.03 12-3.06	Des: 4 ft Min: 2 ft	2 ft
Cross-Section Elements	CTWLTL Width **(8)**		6-4.03	Desirable: 14 ft Minimum: 11 ft	
Cross-Section Elements	Parking Lane Width **(9)**		14-2.06	Desirable: 12 ft Minimum: 7 ft	
Cross-Section Elements	Sidewalk Width		14-2.06	Minimum: 5 ft	
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	*Design Loading Structural Capacity	12-3.03	See Note **(10a)**	
Cross-Section Elements	Reconstructed/ Rehabilitated Bridges	Minimum Width	12-3.03	Approach Roadway Width	
Cross-Section Elements	Existing Bridges to Remain in Place	*Design Loading Structural Capacity	12-3.03	See Note **(11)**	
Cross-Section Elements	Existing Bridges to Remain in Place	Minimum Width	12-3.03	See Note **(11)**	
Cross-Section Elements	Roadside Clear Zone	Guardrail	12-3.03 12-3.08	Usable Shoulder Width	
Cross-Section Elements	Roadside Clear Zone	Lateral Offset	12-3.03 12-3.08	1.5 ft **(12a)**	See Note **(12b)**
Cross-Section Elements	Slope Schedule – Cut	Foreslope	12-3.08 14-2.06	+2%	4:1
Cross-Section Elements	Slope Schedule – Cut	Depth of Ditch	12-3.08 14-2.06	N/A	3 ft
Cross-Section Elements	Slope Schedule – Cut	Backslope	12-3.08 14-2.06	Des: 3:1 Max: 2:1	3:1
Cross-Section Elements	Slope Schedule – Fill	Height ≤ 5 ft	12-3.08 14-2.06	Des: 3:1 Max: 2:1	Des: 4:1 Max: 3:1
Cross-Section Elements	Slope Schedule – Fill	Height > 5 ft	12-3.08 14-2.06	Des: 3:1 Max: 2:1	3:1

	DESIGN SPEED		Manual Section	30 mph	35 mph	40 mph	45 mph	50 mph
Alignment Elements	*Stopping Sight Distance		12-3.05	See Section 12-3.05				
Alignment Elements	Intersection Sight Distance **(13)**		6-6.0	335 ft	390 ft	445 ft	500 ft	555 ft
Alignment Elements	*Superelevation Rate		2-8.02 12-3.04	See Section 12-3.04				
Alignment Elements	*Minimum Horizontal Curve Radius (e_{max} = 6%)		2-8.02 12-3.04	See Section 12-3.04				
Alignment Elements	*Maximum Grades **(14)**	Level	2-8.02 12-3.05	Existing				
Alignment Elements	*Maximum Grades **(14)**	Rolling	2-8.02 12-3.05	Existing				
Alignment Elements	Minimum Grades		12-3.05	See Note **(15)**				
Alignment Elements	*Vertical Curve (K-values)	*Crest	2-8.02 12-3.05	See Section 12-3.05				
Alignment Elements	*Vertical Curve (K-values)	Sag	2-8.02 12-3.05	See Section 12-3.05				
Alignment Elements	*Vertical Clearance (collector under) **(16)**	Reconstructed/ Rehabilitated Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft				
Alignment Elements	*Vertical Clearance (collector under) **(16)**	Existing Bridges	2-8.02	Desirable: 16 ft Minimum: 14.5 ft				
Alignment Elements	*Vertical Clearance (collector under) **(16)**	Sign Truss/ Pedestrian Bridge	2-8.02	19 ft				
Alignment Elements	Vertical Clearance (collector over railroad) **(17)**		2-8.02	Desirable: 25 ft Minimum: 23.5 ft				

*For application of controlling design criteria, see Section 2-9.02.

Table 12-3-D
GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTORS (MULTILANE)
(3R Criteria)

	DESIGN ELEMENT			Manual Section	With Curb (1)	Without Curb (1)
Design Controls	Design Year			14-2.01	Desirable: 10 years Minimum: Current	
	*Design Speed **(2)**			12-3.02	30 mph – 50 mph	
	Control of Access			11-1.05	Control by Regulation (Type 3)	
	Level of Service Threshold			14-2.01	Desirable: C Minimum: D	
Cross-Section Elements	*Travel Lane Width **(3)**			12-3.03	NHS: 12 ft Non-NHS: Des: 12 ft Min: 11 ft	
	Outside Shoulder Width		*Usable **(4)**	12-3.03	Des: 8 ft Min: 2 ft	Des: 10 ft Min: 4 ft
			Paved		Same as Usable	Des: 8 ft Min: 2 ft
	Median Shoulder Width		*Usable **(4)**	12-3.03	Des: 4 ft Min: 2 ft	Des: 8 ft Min: 4 ft
			Paved		Same as Usable	3 ft
	*Cross Slope		Travel Lane **(6)**	2-8.03 12-3.04	2%	
			Shoulder		Shoulder Width > 4 ft: 4% Shoulder Width ≤ 4 ft: 2%	See Note **(7)**
	Auxiliary Lanes		Lane Width	2-8.03 12-3.06	Desirable: Same as Travel Lane Min: 10 ft	
			Usable Shoulder Width		Des: 4 ft Min: 2 ft	2 ft
	CTWLTL Width **(8)**			6-4.03	Desirable: 14 ft Minimum: 11 ft	
	Parking Lane Width **(9)**			14-2.06	Desirable: 12 ft Minimum: 7 ft	
	Sidewalk Width			14-2.06	Minimum: 5 ft	
	Median Width			14-2.06	Existing	
	Reconstructed/ Rehabilitated Bridges		*Design Loading Structural Capacity	12-3.03	See Note **(10a)**	
			Minimum Width		Approach Roadway Width	See Note **(10b)**
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	12-3.03	See Note **(11)**	
			Minimum Width			
	Roadside Clear Zone		Guardrail	12-3.03 12-3.08	Usable Shoulder Width	
			Lateral Offset		1.5 ft **(12a)**	See Note **(12b)**
	Slope Schedule	Cut	Foreslope	12-3.08 14-2.06	+2%	4:1
			Depth of Ditch		N/A	4 ft
			Backslope		Des: 3:1 Max: 2:1	3:1
		Fill	Height ≤ 5 ft		Des: 3:1 Max: 2:1	Des: 4:1 Max: 3:1
			Height > 5 ft			3:1

	DESIGN SPEED		Manual Section	30 mph	35 mph	40 mph	45 mph	50 mph
Alignment Elements	*Stopping Sight Distance		12-3.05	See Section 12-3.05				
	Intersection Sight Distance		6-6.0	See Note **(13)**				
	*Superelevation Rate		2-8.02 14-2.04	See Section 12-3.04				
	*Minimum Horizontal Curve Radius (e_{max} = 6%)		2-8.02 14-2.04	See Section 12-3.04				
	*Maximum Grades **(14)**	Level	2-8.02 12-3.05	Existing				
		Rolling						
	Minimum Grades		12-3.05	See Note **(15)**				
	Vertical Curve (K-values)	*Crest	2-8.02 12-3.05	See Section 12-3.05				
		Sag						
	*Vertical Clearance (collector under) **(16)**	Reconstructed/ Rehabilitated Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft				
		Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft				
		Sign Truss/ Pedestrian Bridge		19 ft				
	Vertical Clearance (collector over railroad) **(17)**		2-8.02	Desirable: 25 ft Minimum: 23.5 ft				

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 12-3-C and 12-3-D

1. *Multilane Roadways – The 3R criteria in the table for multilane roadways should apply to existing multilane roadways only. For an existing 2/3-lane roadway that will become a multilane roadway, the new construction/reconstruction criteria provided in Table 14-2-I should be used.*

2. *Design Speed – A design speed as low as the posted speed limit may be used.*

3. *Travel Lane Width –*

   a. *Existing lane widths on non-NHS routes may be retained in the absence of a crash pattern or if right of way and/or environmental constraints exist.*

   b. *11-foot lane widths may be used on NHS routes in more constrained areas and where truck and bus volumes are low. The lane widths needed for all lanes and intersection design controls should be evaluated collectively with consideration of all user modes and the adjacent land use.*

   c. *The lane width should desirably be increased by two feet for roadways that include angled on-street parking. See Section 14-2.06.06 for more information.*

4. *Shoulder Width –*

   a. *The minimum 2-foot shoulder width may be used on curbed roadways when right of way and/or other constraints exist. For any widths less than the desirable width, the drainage design should include consideration of the shoulder width in order to minimize ponding on the roadway.*

   b. *See Section 14-2.06.01 for information on shoulder width relative to curb and gutter usage.*

5. *Shoulder Surface Type – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved for special conditions.*

6. *Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0%. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

7. *Cross Slope of Shoulder (normal crown sections) –*

   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

8. *CTWLTL Width – In industrial areas with heavy truck traffic, the CTWLTL width should desirably be 16 feet. Section 6-4.03 provides more information on the CTWLTL width in areas of restricted right of way.*

9. *Parking Lanes – The widths shown in the tables only apply to parallel parking. Section 14-2.06.06 provides information on the design of angled parking.*

10. *Reconstructed/Rehabilitated Bridges* –

    a. *Design Loading Structural Capacity should equal HL-93 for reconstructed bridges. For Design Loading Structural Capacity on rehabilitated bridges, see Section 12-2.03.03.*

    b. *The width of reconstructed/rehabilitated bridges should be as follows:*

       ADT ≤ 2000: Traveled way + four feet (each side)
       ADT > 2000: Approach Roadway Width

11. *Existing Bridges to Remain in Place* –

    a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

    b. *The existing bridge width may remain unchanged if there is no crash pattern associated with the existing width. However, consideration should still be given to widening or reconstructing the bridge. If a crash pattern does exist, the bridge should be widened or reconstructed to the approach roadway width.*

    c. *See Section 14-2.06.05 for information on sidewalks at existing bridges.*

12. *Roadside Clear Zones*

    a. *With Curb – Desirably, the clear zone for open shoulder rural sections (Section 9-2.0) should also be provided for curbed sections. The 1.5-foot lateral offset should be measured from the face of the curb to the obstruction, regardless of shoulder width.*

    b. *Without Curb – The clear zone for open shoulder rural sections (Section 9-2.0) should be provided. Where the recommended clear zone distances cannot be met, a 12-foot lateral offset (measured from edge of traveled way) should be provided on the outside of horizontal curves, and an 8-foot lateral offset should be provided elsewhere.*

13. *Intersection Sight Distance (ISD) – The values provided in Table 12-3-C assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways. However, in the absence of a crash pattern, the existing ISD is typically acceptable without further evaluation. If the decision is made to improve the ISD, the improved design should desirably meet the criteria for new construction/reconstruction in Section 6-6.0. At existing intersections, it may not be feasible to remove obstructions within the sight triangle. In such cases, the designer should provide as much sight distance as feasible.*

14. *Maximum Grades – The existing grades may remain unchanged if the project scope does not necessitate regrading of the roadway or if there is no crash pattern associated with the existing grades. However, if either condition exists, the roadway should be regraded to the extent feasible in accordance with the maximum grades listed in the new construction/reconstruction geometric design criteria tables in Chapter 14, "Basic Design Controls".*

15. *Minimum Grades* –

    a. *On roadways, level gradients are acceptable on pavements that are adequately crowned to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    b. *On new or reconstructed bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

16. *<u>Vertical Clearance (collector under)</u> –*

    a. *The vertical clearance shown in the tables should be provided over the entire collector roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the collector roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance, if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

17. *<u>Vertical Clearance (collector over railroad)</u> – If a project includes new or reconstructed bridges, the vertical clearance shown in the tables should typically be provided over the entire railroad right of way width.*

12-4.0 3R FREEWAY PROJECTS

12-4.01 General

12-4.01.01 Background

The freeway system provides a level of mobility and safety for the traveling public that is unattainable without its special features (e.g., full control of access, wide roadway widths, higher design speeds). The freeway system requires periodic maintenance and upgrading that can exceed the limits of normal maintenance. The next level of improvements are referred to as 3R freeway projects (resurfacing, restoration, rehabilitation). Applying new construction/ reconstruction criteria to 3R freeway projects is often infeasible; therefore, any projects that also include geometric improvements as part of the 3R scope of work involves design considerations that are discussed in the following sections.

The next level of maintenance activities falls under preventive maintenance (1R) work. Pavement preventive maintenance projects with work such as pavement sealing, single lift overlays, concrete grinding, and joint repairs fall into the 1R category. See Section 12-6.0 for further details on 1R type projects and design criteria for such work.

12-4.01.02 Objectives

The objective of a 3R freeway project is, within feasible limits, to return the freeway to its original Level of Serviceability or to improve its serviceability to meet current and future demands. This objective applies to all aspects of the freeway's serviceability including, but not limited to:

1. structural adequacy
2. drainage
3. LOS for the traffic volume
4. geometric design
5. roadside safety

12-4.01.03 Approach

3R freeway projects are most often initiated to make a specific improvement to the freeway. The geometric design tables in Section 12-4.02.08 present the criteria for 3R freeway projects. The 3R approach is summarized as follows:

1. Nature of Improvement –Typical improvements include, but are not limited to:

   a. Multiple-lift resurfacing, restoration, and/or rehabilitation of the pavement, including shoulders
   b. improving roadway delineation
   c. adding auxiliary lanes
   d. upgrading roadside safety
   e. increasing the length of acceleration and/or deceleration lanes at an interchange
   f. widening an existing bridge
   g. improving the roadside drainage

2. Other Improvements – Any other geometric design deficiencies within the project limits that can be feasibly improved should be identified. Engineering judgment should be exercised when determining improvements that may be justified.

12-4.01.04 3R Project Evaluation

The various factors outlined in Section 12-2.01 should be evaluated for the design of 3R freeway projects.

12-4.02 Design Controls

Some specific geometric design elements on an existing freeway may have met the applicable design criteria at the time of original construction, but may not meet the Department's current new construction/reconstruction criteria. In these cases, consideration should be given to improving such design elements to meet the current 3R criteria as much as feasible. Additionally, existing design elements along freeways should not be reduced in order to accomplish the goals of a 3R freeway project.

The following subsections outline the design controls for 3R freeway projects.

12-4.02.01 Design Speed

The 3R design speed should match the design speed at the time of the original construction or the posted speed limit, whichever is greater.

12-4.02.02 Highway Capacity

Desirably, the freeway should meet the geometric design criteria for traffic volumes determined for 20 years beyond the expected completion date. However, 10 years beyond the completion date should be used as a minimum.

12-4.02.03 Cross-Section Elements

The geometric design criteria tables in Section 12-4.02.08 present the 3R criteria for the design of 3R rural and urban freeway projects. The information presented in Section 12-2.03 for rural non-freeways also applies to freeways.

12-4.02.04 Vertical Clearances

The minimum vertical clearance for freeways passing beneath a structure should be 16 feet over the entire roadway width, including auxiliary lanes and shoulders. However, a vertical clearance of 17 feet should desirably be provided to allow for future resurfacing with additional structure thickness. The Director of the Surface Deployment and Distribution Command Transportation Engineering Agency (SDDCTEA) should be notified by the Roadway Design Division Engineer about any location on the Interstate System where the minimum vertical clearance is not met. See Section 2-10.03 for more information.

12-4.02.05 Interchanges

A 3R freeway project may include improvements to one or more interchanges. The improvements may be to rehabilitate an entire interchange or to make only selective improvements to the interchange geometrics. Typical improvements at existing interchanges may include, but are not limited to:

- lengthening exit and entrance ramps
- converting ramp terminals to roundabouts
- signalizing ramp terminals

Chapter 7, "Grade Separations and Interchanges", should be referenced for improvements that are proposed at exit and entrance ramps. Chapter 6, "At-Grade Intersections", should be referenced for improvements that are proposed at ramp terminals.

12-4.02.06 Emergency Crossovers

Any existing emergency crossovers and new proposed emergency crossovers should be evaluated for compliance with the provisions within the current AASHTO publication *A Policy on Geometric Design of Highways and Streets* and then, if justified to remain or to be installed, should be in conformance with the current MDOT Standard Drawing for Emergency Crossovers. This evaluation may result in the need for not only adjusting existing crossovers but possibly also the removal of existing crossovers altogether.

12-4.02.07 Roadside Safety

One of the objectives of a 3R freeway project may be to upgrade roadside safety along the freeway, as discussed in the following subsections. Chapter 9, "Roadside Safety", should be referenced for additional information.

12-4.02.07.1 *Side Slopes*

For non-interstate freeways, the following recommendations should be followed for existing side slopes within the clear zone:

- <u>Side Slopes 4:1 or Flatter</u> – A 30-foot clear zone should be provided. Flattening of slopes or providing guardrail is typically not warranted because these slopes are considered recoverable.

- <u>Side Slopes Between 3:1 (Inclusive) and 4:1 (Exclusive)</u> – A 30-foot clear zone should be provided. Slopes between 3:1 (inclusive) and 4:1 (exclusive) are typically acceptable because such slopes are considered traversable; however, if a safety analysis indicates a crash pattern, consideration should be given to flattening the side slopes, providing guardrail, and/or providing other remedial actions.

- <u>Side Slopes Steeper than 3:1</u> – Side slopes steeper than 3:1 should be flattened when feasible, or guardrail should be provided, because these slopes are considered non-traversable. If the side slopes are flattened, the above recommendations should be followed.

12-4.02.07.2 *Cross Drain Structures*

The following information should be followed for existing cross drain structures:

1. Beyond Clear Zone – Cross drain structures with headwalls or parapets located beyond the clear zone should typically not warrant treatment.

2. Within Clear Zone – Cross drain structures with headwalls or parapets within the clear zone should be treated as follows:

   a. If the opening is 36 inches or less, any headwall should be replaced with a flared end section. No other treatment is warranted for such pipes that have flared ends. However, if the project includes flattening the side slopes, the cross drain should be extended beyond the clear zone.

   b. If the opening is greater than 36 inches, the drainage structure should be extended beyond the clear zone or guardrail should be installed, regardless of whether the project includes flattening the side slopes.

   c. Where an existing drainage structure is to be extended on a project that does not otherwise include flattening of the side slopes, a minimum transverse slope of 6:1 should be provided on the approach side within the clear zone. See Section 9-3.06 for more information.

12-4.02.07.3 *Medians*

The following information should be followed for obstacles and transverse slopes in the median:

1. Median Width 64 feet and Wider – Obstacles (e.g., bridge piers, drainage structures) that are located outside of the clear zone will typically not warrant treatment.

2. Median Width Less than 64 feet – Any obstacles within the clear zone should be evaluated for treatment. If the existing median side slopes are 10:1 or flatter, a barrier should be placed as close to the obstacle as feasible, subject to the dynamic deflection of the system. If the side slopes are steeper than 10:1, the following options are available:

   a. The median slope can be regraded to 10:1 and a barrier be placed next to the obstacle, which is the more desirable option. However, drainage considerations may render this option infeasible.

   b. A barrier can be installed with the face of the barrier to be placed at the outside edge of the normal shoulder line. See *Standard Drawings* for more details.

3. Ditch Plugs – Where the median width is 64 feet or less and median or cross drain pipes are used in conjunction with ditch plugs, the following should be evaluated:

   a. Ditch plugs and inlets should meet current standards.

   b. For pipe sizes greater than 36 inches, drainage inlets should be used that conform to the side slope.

4. Transverse Slopes – Any slopes that are approximately perpendicular to the direction of traffic (e.g., slopes at an emergency crossover) that are steeper than 10:1 within the clear zone should be regraded to 10:1 or flatter.

12-4.02.07.4 *Exit and Entrance Gores*

The gore area normally includes the paved triangular area between the through lane and the exit or entrance lane, plus the graded area that may extend downstream or upstream from the gore nose. The following information should be followed when evaluating the gore area:

1. Obstacles – The area beyond the exit gore nose should desirably be kept free of all obstacles (except the ramp exit sign). Existing breakaway supports should be checked to ensure that they are functional. In addition, any curbs and exposed sign footings should be removed from both entrance and exit gores. Any other obstacles within the clear recovery area of the gore nose should be made breakaway or shielded by a barrier or impact attenuator.

2. Side Slopes – The graded area beyond or before the gore nose should be as flat as feasible. If the elevation between the exit or entrance ramp and the mainline increases rapidly, a relatively flat graded area may not be feasible. These areas may be non-traversable and the gore design should shield the driver from these areas. At some sites, the vertical divergence of the ramp and mainline may warrant protection for both roadways beyond or before the gore.

3. Cross Slopes – The paved triangular gore or neutral area between the through lane and exit ramp should be safely traversable. See the *Standard Drawings* for proposed cross-slopes in the gore area.

4. Drainage – When proposed, drainage inlets should normally be placed between the physical nose and the gore nose. The presence of drainage inlets may result in two breaks in the gore cross slope. Drainage design should avoid sheet flow across the travel lanes from adjacent ditches.

12-4.02.07.5 *Safety Appurtenances*

Section 12-2.08 presents information on safety appurtenances, which apply to 3R freeway projects.

12-4.02.08 3R Tables of Geometric Design Criteria

Table 12-4-A and Table 12-4-B present the geometric design criteria for rural and urban 3R freeway projects, respectively.

Each table is followed by a set of footnotes that provides additional information for the items listed in the criteria tables.

When the design criteria proposed for 3R freeway projects do not comply with the values presented in the above-referenced tables, see Design Exception and Design Variance procedures presented in Section 2-10.0.

The criteria provided in these tables, as well as the additional information provided in the footnotes for this table, meet or exceed the criteria in the AASHTO publication *A Policy on Design Standards – Interstate System*. The design for all interstates shall be in compliance with these criteria.

Table 12-4-A
GEOMETRIC DESIGN CRITERIA FOR RURAL FREEWAYS
(3R Criteria)

	DESIGN ELEMENT			Manual Section	Rural
Design Controls	Design Year			12-3.02	Desirable: 20 Years Minimum: 10 Years
	*Design Speed			12-3.02	See Note **(1)**
	Control of Access			11-1.05	Full (Type 1)
	Level of Service Threshold			2-3.04	B
Cross-Section Elements	*Lane Width			2-8.03	12 ft
	Outside Shoulder Width **(2)**		*Usable	2-8.03	12 ft
			*Paved		10 ft
	Median Shoulder Width **(3)**		*Usable	2-8.03	8 ft
			*Paved		4 ft
	*Cross Slope		Travel Lane **(3)**	2-8.03	2%
			Shoulder		See Note **(4)**
	Auxiliary Lanes		Lane Width	2-8.03	12 ft
			Shoulder Width		Paved: 10 ft Usable: 12 ft
	Median Width		Depressed	2-8.03	Existing
			Concrete Median Barrier		N/A
	Reconstructed/ Rehabilitated Bridges		*Design Loading Structural Capacity	2-8.03 12-4.02	See Note **(5)**
			Minimum Width		Traveled Way + 12 ft (outside shoulder) + 6 ft (median shoulder)
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	2-8.03 12-4.02	See Note **(6)**
			Minimum Width **(6)**		Traveled Way + 10 ft (outside shoulder) + 4 ft (median shoulder)
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			2-8.03 11-1.01	Existing
	Roadside Clear Zone		Guardrail	2-8.03 9-2.0 12-4.02	Usable Shoulder Width
			Obstruction **(7)**		30 ft
	Slope Schedule **(8)**	Cut	Foreslope (within clear zone)	2-8.03 12-4.02	Existing
			Depth of Ditch		Existing
			Backslope		Existing
		Fill	Safety Slope (within clear zone)		Desirable: 6:1 Maximum: 3:1
			Fill Slope (outside clear zone)		Maximum: 2:1
Alignment Elements	**DESIGN SPEED**				**70 mph**
	*Stopping Sight Distance			2-8.02	Existing
	Decision Sight Distance **(9)**			2-8.02	1105 ft
	*Superelevation Rate			2-8.02 3-4.01	See Note **(10)**
	*Minimum Horizontal Curve Radius			2-8.02 3-3.0	Existing
	*Maximum Grades	Level		2-8.02 12-2.05	Existing
		Rolling			Existing
	Vertical Curve (K-values)		*Crest	2-8.02 4-5.0	Existing
			Sag		
	*Vertical Clearance (freeway under) **(11)**		Rehabilitated Bridges	2-8.02 12-4.02	Desirable: 17 ft Minimum: 16 ft
			Existing Bridges		Desirable: 17 ft Minimum: 16 ft
			Sign Truss/ Pedestrian Bridge		19 ft
	Vertical Clearance (freeway over railroad) **(12)**			2-8.02	Desirable: 25 ft Minimum: 23.5 ft

*For application of controlling design criteria, see Section 2-9.02.

Table 12-4-B
GEOMETRIC DESIGN CRITERIA FOR URBAN FREEWAYS
(3R Criteria)

	DESIGN ELEMENT			Manual Section	Urban		
Design Controls	Design Year			12-3.02	Desirable: 20 Years Minimum: 10 Years		
	*Design Speed			12-3.02	See Note **(1)**		
	Control of Access			11-1.05	Full (Type 1)		
	Level of Service Threshold			2-3.04	Desirable: B Minimum: C		
Cross-Section Elements	*Lane Width			2-8.03	12 ft		
	Outside Shoulder Width **(2)**	*Usable		2-8.03	12 ft		
		*Paved			10 ft		
	Median Shoulder Width **(2)**	*Usable		2-8.03	8 ft		
		*Paved			4 ft		
	*Cross Slope	Travel Lane **(3)**		2-8.03	2%		
		Shoulder			See Note **(4)**		
	Auxiliary Lanes	Lane Width		2-8.03	12 ft		
		Shoulder Width			Paved: 10 ft Usable: 12 ft		
	Median Width	Depressed		2-8.03	Existing		
		CMB			≥ 4 Lanes: 26 ft Desirable; 22 ft Minimum		
	Reconstructed/ Rehabilitated Bridges	*Design Loading Structural Capacity		2-8.03 12-4.02	See Note **(5)**		
		Minimum Width			Traveled Way + 12 ft (outside shoulder) + 6 ft (median shoulder)		
	Existing Bridges to Remain in Place	*Design Loading Structural Capacity		2-8.03 12-4.02	See Note **(6)**		
		Minimum Width **(6)**			Traveled Way + 10 ft (outside shoulder) + 4 ft (median shoulder)		
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			11-1.01 14-2.06	Existing		
	Roadside Clear Zone	Guardrail		2-8.03 9-2.0 12-4.02	Usable Shoulder Width		
		Obstruction **(7)**			30 ft		
	Slope Schedule **(8)**	Cut	Foreslope (within clear zone)	2-8.03 12-4.02	Existing		
			Depth of Ditch		Existing		
			Backslope		Existing		
		Fill	Safety Slope (within clear zone)		Desirable: 6:1 Maximum: 3:1		
			Fill Slope (outside clear zone)		Maximum 2:1		
Alignment Elements	**DESIGN SPEED**				**60 mph**	**65 mph**	**70 mph**
	*Stopping Sight Distance			2-8.02	Existing		
	Decision Sight Distance **(9)**			2-8.02	1280 ft	1365 ft	1445 ft
	*Superelevation Rate			2-8.02 3-4.01	See Note **(10)**		
	* Minimum Horizontal Curve Radius			2-8.02 3-3.0	Existing		
	*Maximum Grades	Level		2-8.02 12-3.05	Existing		
		Rolling			Existing		
	Vertical Curve (K-values)	*Crest		2-8.02 4-5.0	Existing		
		Sag					
	*Vertical Clearance (freeway under) **(11)**	Rehabilitated Bridges		2-8.02 12-4.02	Desirable: 17 ft Minimum: 16 ft		
		Existing Bridges			Desirable: 17 ft Minimum: 16 ft		
		Sign Truss/ Pedestrian Bridge			19 ft		
	Vertical Clearance (freeway over railroad) **(12)**			2-8.02	Desirable: 25 ft Minimum: 23.5 ft		

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 12-4-A and 12-4-B

1. *Design Speed – The 3R design speed should match the design speed at the time of the original construction or the posted speed limit, whichever is greater. For an existing freeway that will be widened to provide additional capacity, the new freeway should be designed according to the criteria in Table 2-9-A (rural) or Table 14-2-E (urban).*

2. *Shoulder Width –*

   a. *The minimum paved shoulder widths shown in Table 12-4-A and Table 12-4-B are consistent with the AASHTO publication* A Policy on Design Standards – Interstate System*. Paved shoulder widths on interstates shall not be less than these widths. The table below includes these widths, but also provides additional information regarding usable shoulder widths, freeways with three or more lanes in one direction, and truck traffic.*

No. of Lanes in One Direction	Left		Right	
	Usable	Paved	Usable	Paved
2	8 ft	4 ft	12 ft*	10 ft*
3 or more	12 ft*	10 ft*	12 ft*	10 ft*

Where truck traffic exceeds 250 DDHV, consideration should be given to increasing these widths by two feet.

   b. *Outside of the typical 2-foot paved portion, paved shoulders should not be provided on non-interstate freeways except as approved for special conditions.*

   c. *Where the two roadways are separated by a CMB, the desirable paved median shoulder width is 12 feet; the minimum paved median shoulder width is 10 feet. Where the truck traffic exceeds 250 DDHV, the 12-foot width should be used.*

   d. *A minimum paved shoulder width of 10 feet, desirably 12 feet, should desirably be provided for a distance of 350 feet beyond a reduction of the basic number of lanes (lane drop) of a multilane arterial for a recovery area for drivers. See Section 7-2.05 for more information.*

   e. *An existing usable shoulder width of 10 feet may be retained on non-interstate freeways if right of way and/or other constraints exist.*

3. *Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0% and shall not be less than 1.5%. For freeways with three or more lanes sloped in the same direction, the cross slope may be increased to 2.5% for all lanes beyond the first two lanes that are sloped in the same direction. The cross slope of an outer lane shall not be less than that of the adjacent lane.*

4. *Cross Slope of Shoulder (normal crown sections) – The cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%. The portion of the shoulder that is not paved should have a cross slope of 4%. The cross slope of the shoulder shall not be less than that of the adjacent lane.*

5. <u>Reconstructed/Rehabilitated Bridges</u> –

a. Design Loading Structural Capacity should equal HL-93 for reconstructed bridges. For Design Loading Structural Capacity on rehabilitated bridges, see Section 12-2.03.03.

b. The bridge width shown in the table applies to a 1-way bridge of a divided freeway. The bridge width should be increased for each additional lane and for wider shoulders, as shown in Footnote 2 above.

c. For bridges less than or equal to 200 feet in length, the bridge width should be no less than the full paved width of the approach roadway when the minimum widths provided in the tables cannot be met.

6. <u>Existing Bridges to Remain in Place</u> –

a. Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

b. The width provided in the tables applies to a 1-way existing bridge that is less than or equal to 200 feet in length. For existing 1-way bridges that are greater than 200 feet in length, the minimum shoulder width shall be no less than four feet for both the left and the right sides.

c. For existing bridges that are in compliance with the applicable widths as outlined in the tables or in footnote 7b, consideration should be given to widening the bridges to comply with the new construction width, especially in areas with high traffic volumes.

7. <u>Roadside Clear Zone</u> – The widths shown in the tables are for design speeds of 60 miles per hour to 70 miles per hour, for 6:1 fill slopes, and for an ADT greater than 6000. Section 9-2.0 provides clear zone distances for other roadside conditions. All values are measured from the edge of traveled way, except where long (greater than 0.5 miles) auxiliary lanes are present. On interstates, clear zones should be evaluated, but such improvements are not expected to be included in interstate maintenance projects. However, mitigation efforts, when determined necessary, should include guardrail or other applicable barrier system when recommended clear zone is not provided.

8. <u>Safety Slope</u> – On non-interstate freeways, Existing fill slopes within the clear zone that are steeper than 3:1 should be flattened or guardrail should be provided. On interstates, side slopes should be evaluated, but such improvements are not expected to be included in interstate maintenance projects. However, mitigation efforts, when determined necessary, should include guardrail or other applicable barrier system when side slopes are steeper than 3:1.

9. <u>Decision Sight Distance (DSD)</u>:

a. <u>Rural</u> – The distance shown in Table 12-4-A is for a speed/path/direction change on a rural freeway. Table 4-5-C should be referenced for other applications.

b. <u>Urban</u> – The distances shown in Table 12-4-B is for a speed/path/direction change on an urban freeway. Table 4-5-C should be referenced for other applications.

10. <u>Superelevation Rate</u> – Normally, the existing superelevation rate should be adequate. However, the Standard Drawings should be reviewed to determine if any improvements should be considered.

11. *Vertical Clearance (freeway under) –*

    a. *The vertical clearance shown in the table should be provided over the entire freeway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the freeway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

12. *Vertical Clearance (freeway over railroad) – If a project includes new or reconstructed bridges, the vertical clearance shown in the tables should typically be provided over the entire railroad right of way width.*

12-5.0 SPOT IMPROVEMENTS (NON-FREEWAYS)

12-5.01 Objectives

Spot improvements are intended to correct an identified deficiency at an isolated location on non-freeways. This project scope of work is consistent with the Department's responsibility to provide a safe driving environment for the motoring public that is free of unexpected demands on the driver. Experience has demonstrated the benefits of improving limited roadway sections or spot locations with recognized geometric deficiencies to at least a level consistent with the adjacent roadway sections, thereby providing drivers with a facility that is consistent with the principles of driver expectancy.

The deficiency that the spot improvement should correct may be related to structural, geometric, safety, drainage, or traffic control. These projects are not intended to provide a general upgrading of the roadway, as are projects categorized as new construction/ reconstruction. For these reasons, a flexible approach is necessary to determine the appropriate geometric design criteria that apply to the spot improvement.

Spot improvement projects may also be affected by special criteria that may apply to a particular funding category. For example, roadway safety improvement projects are intended to provide cost-efficient improvements to sites identified in the Department's Strategic Highway Safety Plan. Typical projects include intersection improvements, flattening a horizontal curve, installing roadside barriers, or installing traffic control devices. The Highway and Rail Safety Division is responsible for conducting a preliminary evaluation of the site and recommending improvements. When roadway work is involved, the Roadway Design Division is typically responsible for preparing the detailed project design.

12-5.02 Approach

The Department has adopted a flexible approach to the geometric design of spot improvement projects. The following information summarizes the approach:

1. Design Criteria – The proposed design criteria for the project should reflect the level of improvement to be used to upgrade the roadway. For reconstruction, the criteria contained throughout the manual that apply to reconstruction should be used. The criteria in Section 12-2.0 and Section 12-3.0 apply for 3R improvements to rural and urban non-freeways, respectively.

2. Design Speed – The design speed should be selected based on the adjacent sections for the spot improvement; however, a design speed that is less than the posted speed should not be selected.

3. Application – Any geometric design deficiencies within the project limits should be considered for improvement.

4. Exceptions – See Section 2-10.0 for a discussion on Design Exceptions and Variances to geometric design criteria.

12-6.0 PREVENTIVE MAINTENANCE (1R) FOR ALL ROADWAY CLASSIFICATIONS

Preventive maintenance projects are primarily intended to extend the service life of the pavement and to enhance roadway safety; therefore, right of way acquisition typically will not be required. Additionally, existing design elements along freeways should not be reduced in order to accomplish the goals of a 1R freeway project.

Improvements for 1R projects may include, but are not limited to:

1. seal coats (fog seals, chip seals, scrub seals, slurry seals)
2. single lift overlays (2-inch maximum lift thickness (milled or non-milled))
3. ultra-thin/thin lift overlays
4. open-graded friction course (OGFC)
5. pre-leveling
6. trench widening
7. diamond grinding
8. punchouts
9. joint/spall repairs
10. dowel bar repairs
11. placement of rumble strips/stripes
12. upgrading guardrail and other safety devices to meet current criteria
13. upgrading signs
14. safety improvements to existing bridges
15. upgrading curb ramps to *ADA* compliance (if project is classified as an alteration)
16. replacement to *ADA* standards of existing pedestrian facilities affected by construction
17. railroad/roadway crossings improvements

Where these minor alterations are proposed to existing roadways such that the project results in no impacts to the pedestrian facilities, curb ramps are the only feature of the pedestrian facilities that shall be reconstructed to the current criteria. These treatments serve solely to seal and protect the roadway surface, improve friction, and control splash and spray are considered to be maintenance because they do not significantly affect the public's access to or usability of the road.

Crosswalks constitute distinct elements of the right-of-way intended to facilitate pedestrian traffic. Regardless of whether there is curb-to-curb resurfacing of the roadway in general, resurfacing of a crosswalk also requires the provision of curb ramps at that crosswalk.

See the Department's "Pavement Preservation & Preventive Maintenance Treatment Policy for Federal-Aid Projects" for more information.

12-7.0 REFERENCES

1. Title 23, *Code of Federal Regulations*, Part 625.

2. Federal Register, "Design Standards for Highways; Resurfacing, Restoration and Rehabilitation of Streets and Highways Other Than Freeways," June 10, 1982.

3. FHWA Technical Advisory T5040.28, "Developing Geometric Design Criteria and Processes for Non-Freeway RRR Projects," FHWA, 1988.

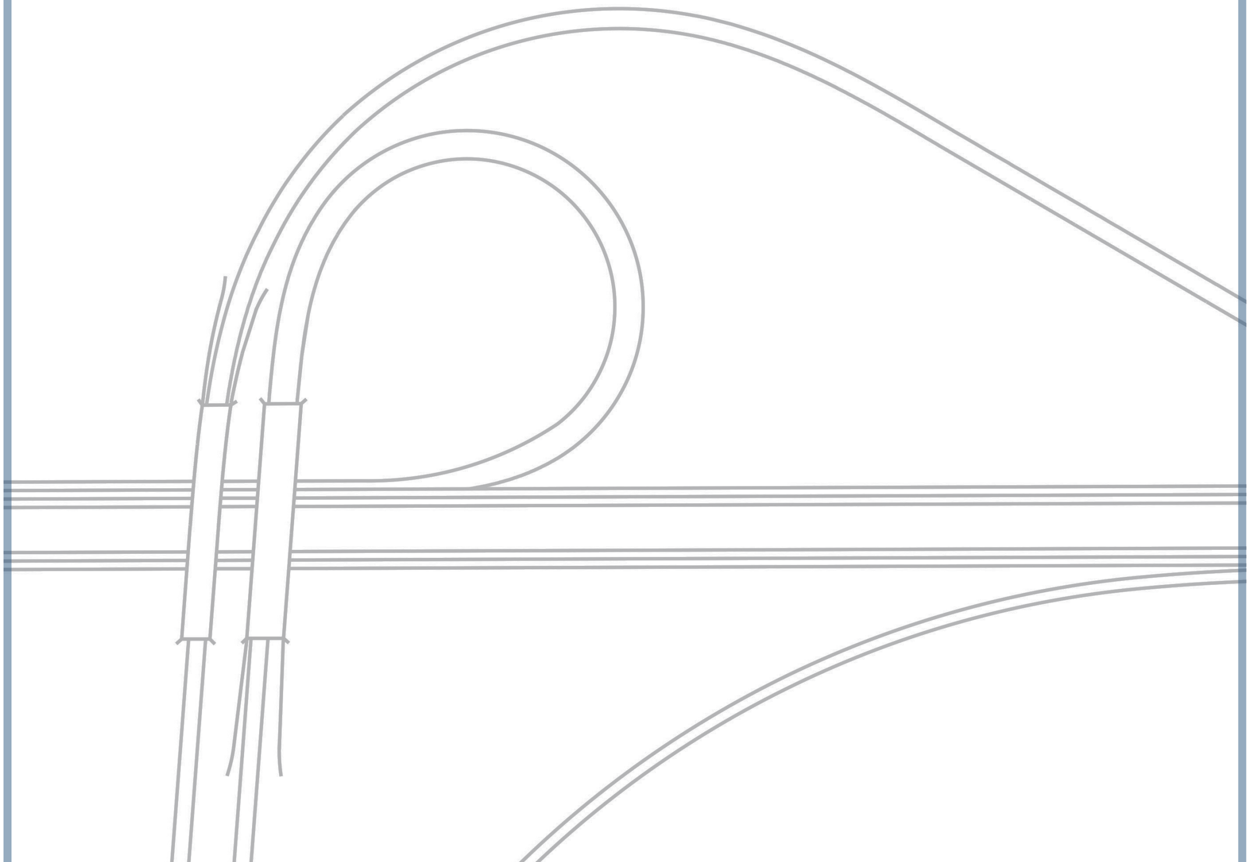
4. *Designing Safer Roads; Practices for Resurfacing, Restoration and Rehabilitation*, Special Report 214, Transportation Research Board, 1987.

5. NCHRP Synthesis 417 *Geometric Design Practices for Resurfacing, Restoration, and Rehabilitation*, Transportation Research Board, 2011.

6. *Mitigation Strategies for Design Exceptions*, FHWA, 2007.

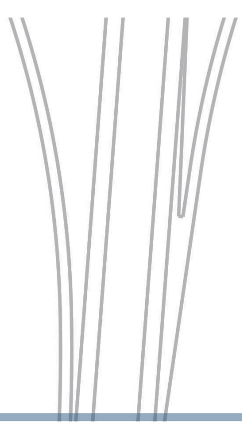
7. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2018.

8. *Roadside Design Guide*, AASHTO, 2011.



CHAPTER 13

Pavement Design



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Chapter 13
PAVEMENT DESIGN

This chapter addresses procedures for designing pavement structures and estimating quantities of pavement structure components for Department projects.

13-1.0 OVERVIEW OF PAVEMENT DESIGN PROCESS

Responsibilities for the design of pavement structures are shared jointly by the District Materials Engineer and the Roadway Design Division Pavement Engineer. The typical procedures for the design of pavement structures are described below:

1. Prior to beginning the development of Contract Plans, Planning Division should provide updated traffic data at the request of the Roadway Design Division Pavement Engineer, who should then request a pavement recommendation from the District Materials Engineer. A current set of plans should be included in this request.

2. For projects that are expected to generate a significant amount of earthwork, the Roadway Design Division Section Engineer should provide a set of conceptual plans (approximately 30% complete) to the District Office to assist them in preparing a soil profile and earthwork recommendations in accordance with S.O.P. Number TMD-20-14-00-000. District recommendations should be provided prior to the completion of Field Inspection Plans to allow sufficient time for implementation into the plan set.

3. For projects that are split into separate grading and paving projects, the pavement recommendation for the mainline is not needed for construction of the grading project. However, the District should recommend a preliminary pavement structure thickness so that plans may be developed and the project graded for the estimated structure thickness. A pavement recommendation should be submitted, though, for any local roadways that are reconstructed as part of the grading project. The District should conduct an additional soil investigation and prepare a final design soil profile after the grading project has been completed. When plan development for the paving project is ready to begin, the pavement design process described above should begin.

4. For projects that include both grading and paving, the District should submit earthwork and pavement recommendations based on the original soil profile. After grading is complete, the District should submit a final soil profile. If changes are necessary in the pavement design, the District should submit a revised pavement recommendation to the Pavement Engineer for review with the Pavement Committee. If approved, a revised pavement design will be issued.

5. For projects that do not include a significant amount of earthwork, or for projects that are on existing alignment, the District may submit a pavement recommendation without an earthwork recommendation. However, earthwork recommendations should be considered for small projects (e.g., bridge replacement projects) that are on new alignment.

6. Pavement design submittals are typically not required for non-Interstate projects involving pavement work located outside the travel lanes.

7. The Pavement Engineer will prepare a draft pavement design based on this pavement recommendation, and will review the design with the Pavement Committee.

8. After appropriate changes have been incorporated, the final pavement design will be submitted to the Chief Engineer for approval or, if applicable, to FHWA for their review and approval.

9. The designer is provided with a copy of the approved pavement design and should prepare typical sections and plan grades to accommodate the pavement structure. The designer should also calculate estimated quantities based on the approved pavement design.

13-2.0 PAVEMENT DESIGN PROCEDURE

13-2.01 Pavement Design Background

The total pavement structure is comprised of various thicknesses of the structural components. The principal factors that influence the thickness of the pavement structure are traffic volumes, percent of trucks, weight of trucks, and soil characteristics of the roadbed.

The method of determining the thickness of each component is described in "Pavement Design Systems for Mississippi Highways," Final Report, State Study No. 66, September 1983, and the current edition of AASHTO's *Guide for Design of Pavement Structures and Supplement* thereto. The acceptable asphalt lift thicknesses are outlined within Section 13-2.03.01.

The "MDOT Pavement Design Procedures" should be referenced for additional information on pavement design.

13-2.02 Pavement Structure Components

The Department recognizes three basic components of a pavement structure:

- pavement
- base course
- design soil

The various alternatives for each of the basic components are described in the following sections.

13-2.02.01 Pavement

The pavement is the top component of a pavement structure on which vehicles travel. There are two standard types of pavement:

- asphalt pavement (flexible)
- Portland cement concrete pavement (rigid)

13-2.02.02 Base Course

The component immediately below the pavement is the base course. The four most common base courses are:

1. granular base course
2. chemically treated base course
3. crushed stone/concrete
4. asphalt pavement

The granular base course is typically limited to lower traffic volume roads. An asphalt base is typically used under Portland cement concrete pavement. Crushed stone/concrete or treated granular base courses may be specified under both of the pavement types.

13-2.02.03 Design Soil

The design soil prism is defined as that portion of the roadbed consisting of the three feet of soil immediately below the base course. This soil may be undisturbed natural earth strata in areas of excavation or embankment constructed with unclassified excavation or borrow excavation.

The design soil treatment is important in the design of pavement structures, but it is not considered a part of the total pavement structure.

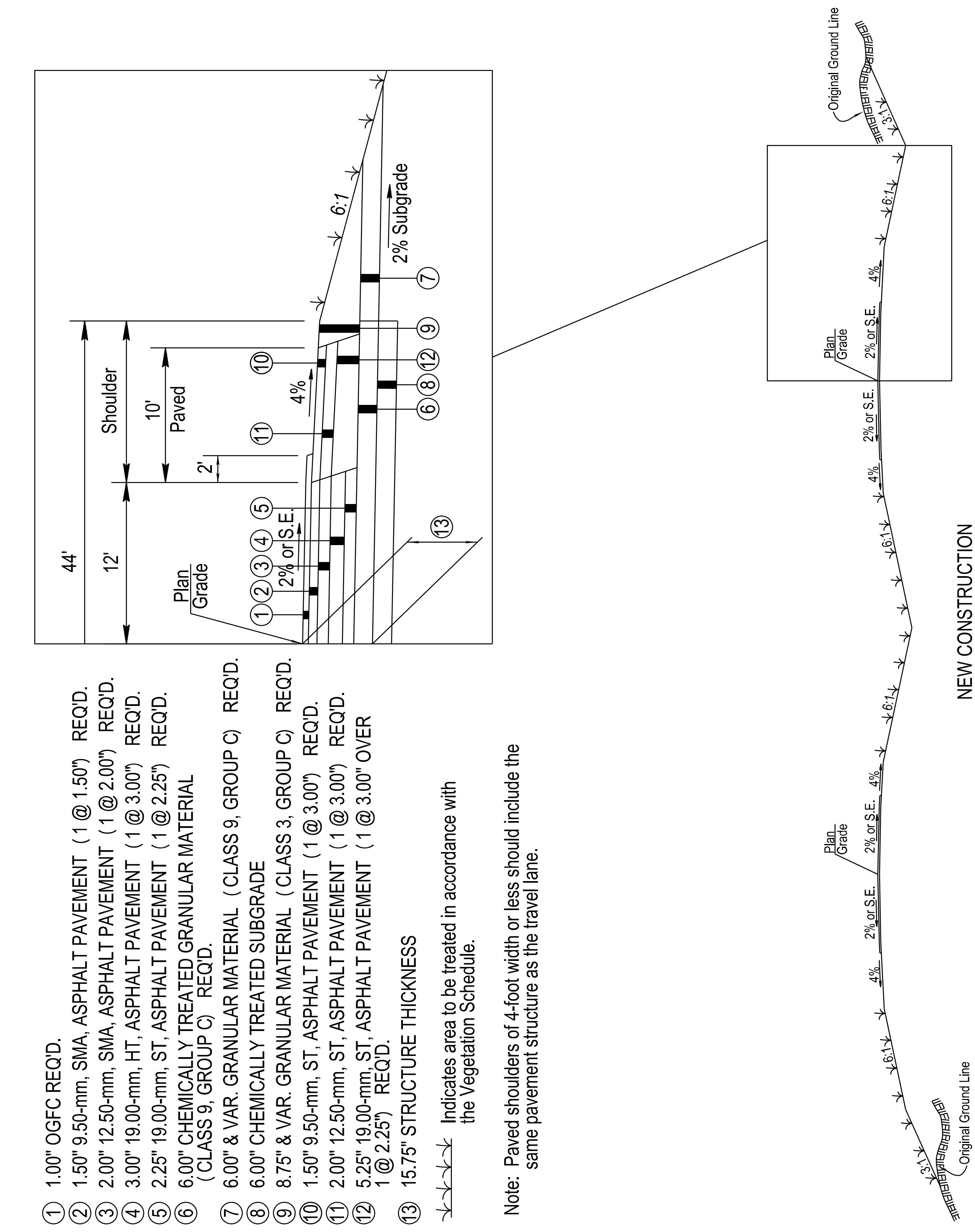
It may be advantageous for the design soil to be fully controlled, depending on the existing soil conditions and the availability of a higher quality borrow material. To control the design soil prism, the District Materials Engineer is responsible for recommending the class of borrow material to be used for the construction of the design soil prism in both cut and fill sections. Removal of the top three feet below the subgrade may be recommended if material in the cut sections does not meet the desired specifications.

If high-volume change (HVC) soil (i.e., VC > 60%) exists in the design soil prism, an acceptable method in S.O.P. Number TMD-20-14-00-000 should be chosen to treat the material.

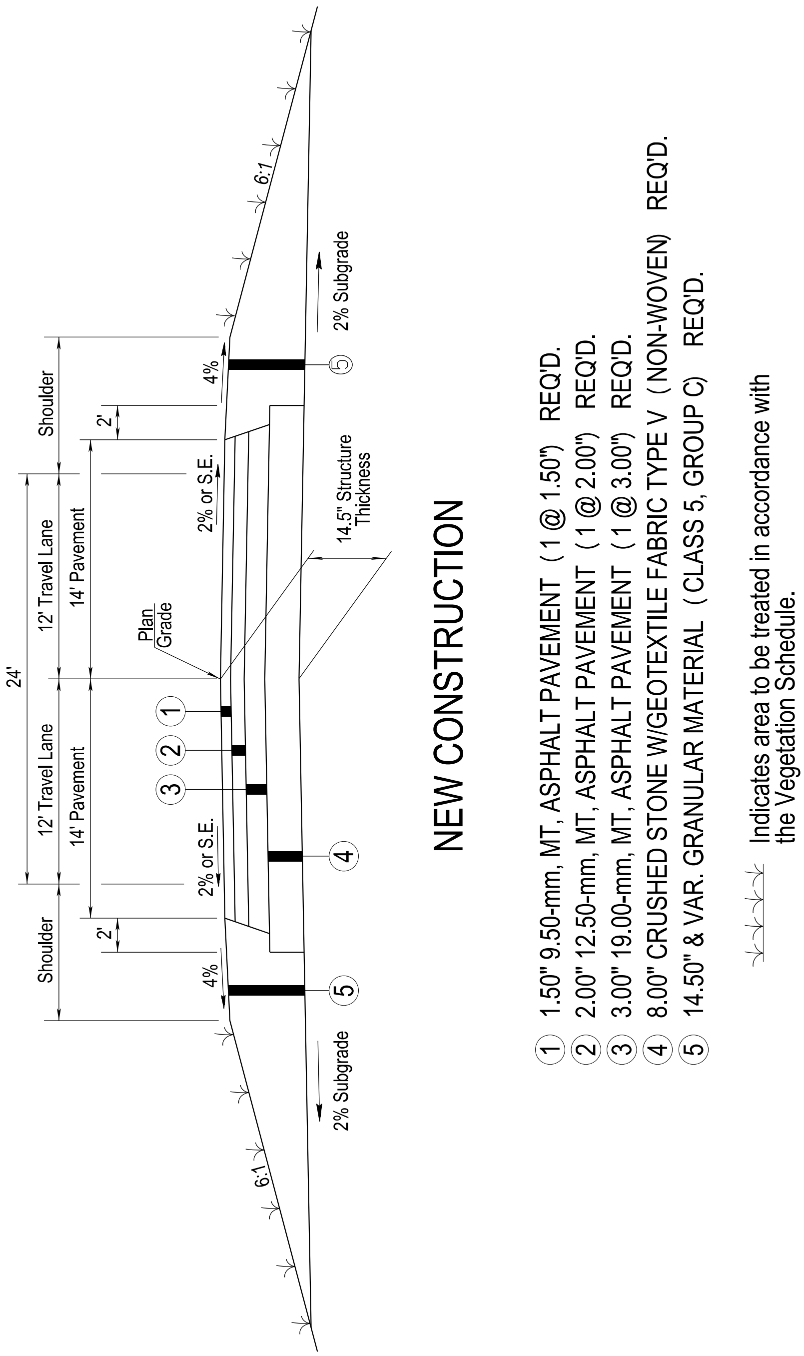
The Department may also elect to chemically treat non-HVC subgrades in order to enhance constructability.

13-2.03 <u>Miscellaneous Typical Pavement Structure Details</u>

The design details of the pavement structure courses should be clearly identified in the plans. Typical sections should be prepared showing the thickness and widths of pavement to be placed. A detail of each pavement type is illustrated in Figures 13-2-A and 13-2-B.



TYPICAL PAVEMENT DETAILS
(Four-Lane New Construction)
Figure 13-2-A



TYPICAL PAVEMENT DETAILS
(Two-Lane New Construction)
Figure 13-2-B

13-2.03.01 Layers of Asphalt Pavement

Asphalt pavement is normally designed and constructed using multiple lifts of asphalt. Each lift is defined by one of the five maximum nominal size aggregates (i.e., 4.75 mm, 9.5 mm, 12.5 mm, 19.0 mm, 25.0 mm) and one of the six mix designs (i.e., Standard Type (ST), Medium Type (MT), High Type (HT), Polymer Modified High Type, Stone Matrix Asphalt (SMA), Open Graded Friction Course (OGFC)). Each layer should be identified separately on the typical sections because each layer is measured and paid for separately. Acceptable lift thicknesses are shown in Table 13-2-A.

Table 13-2-A
ACCEPTABLE LIFT THICKNESS

Maximum Nominal Aggregate Size (mm)	Minimum Lift Thickness (inch)	Maximum Lift Thickness (inch)
4.75	0.5	0.75
OGFC	1	1
9.5	1	1½
12.5	1½	2½
19.0	2¼	3½
25.0	3	4

13-2.03.02 Mix Designations

A mix type designation map for all MDOT routes is published by the Planning Division and can be accessed under the Quick Links of Planning Division's intranet webpage. The mix type designations are based on the number of 80-kip ESALs in the 10-year design life. This map should be used to determine the type of mix to be used on each project.

Flexible pavement designs on interstate routes should include one lift of OGFC and two lifts of SMA as the top three lifts of the pavement structure. The mix designation for other routes should be dictated by the map.

13-2.03.03 Separate Grading and Paving Projects

In some cases, separate projects may be programmed to complete the ultimate facility, including:

- a grading and drainage project (i.e., with mainline paving planned as a future project)
- a paving project that includes the construction of a pavement structure with grading having been completed on a previous project

The typical sections should clearly illustrate the details of the work to be included in the current contract. For grading and drainage projects, the future pavement should be shown with a dashed line to identify the estimated thickness of the ultimate pavement structure. For paving projects, the detailed descriptions of the pavement structure should be shown, and the existing embankment should be shown with dashed lines.

13-2.03.04 Supplemental Roadways

In addition to providing typical sections of the mainline, separate typical sections should be included for paving of supplemental roadways such as:

1. interchange ramps
2. frontage roads
3. intersecting roads, including channelization
4. detours
5. extra width at guardrail

Because soil conditions and traffic levels may vary, the details of the pavement structure may not be consistent throughout the project length. Thicknesses may vary and special treatment of unsuitable material may be specified for some sections. Each different situation should be represented by a typical section that clearly indicates the design details and applicable limits by stationing.

13-2.03.05 Safety Edge

In accordance with the Department's Safety Edge Policy, a safety edge should be included on all roadway projects that include asphalt pavement with open shoulders. The appropriate safety edge detail(s) as shown in Figure 13-2-C should be shown on the typical section sheets near the Angle of Repose detail. In addition, the following guidelines should be noted:

1. A safety edge should not be used for curb and gutter sections.

2. Where two lifts (excluding OGFC) or less of asphalt pavement are proposed, a safety edge should be included on each lift.

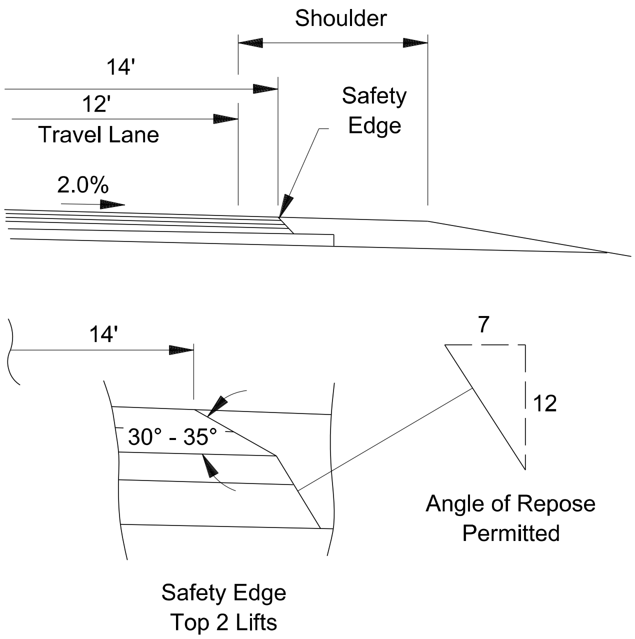
3. Where three lifts (excluding OGFC) or more of asphalt pavement are proposed, a safety edge should be provided on the top two lifts.

4. On roadways with narrow travel lanes and/or narrow shoulders, a minimum one-foot width of trench widening should be included (preferably 2.5 feet to allow for two-foot paved shoulders) prior to adding the safety edge. However, if conditions prevent the trench widening, a safety edge should still be added at the edge of pavement.

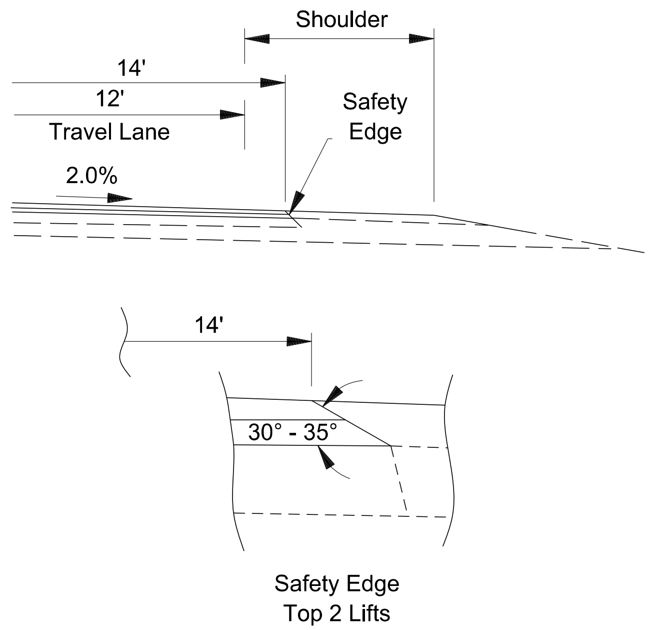
5. The cost of a safety edge is included in the cost of asphalt (i.e., no separate pay item for the safety edge).

6. On "proposal only" projects that have no plans, a safety edge should be provided.

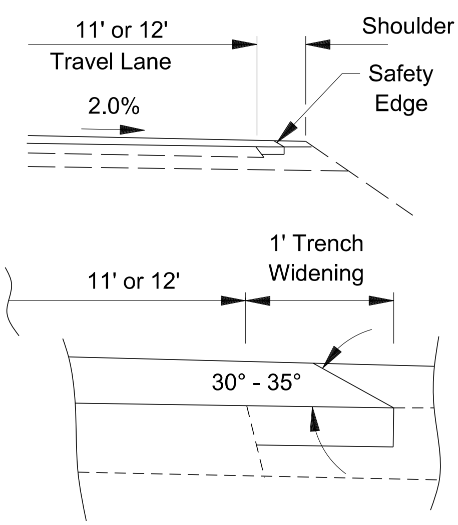
7. A safety edge should also be provided on full-width paved shoulders.



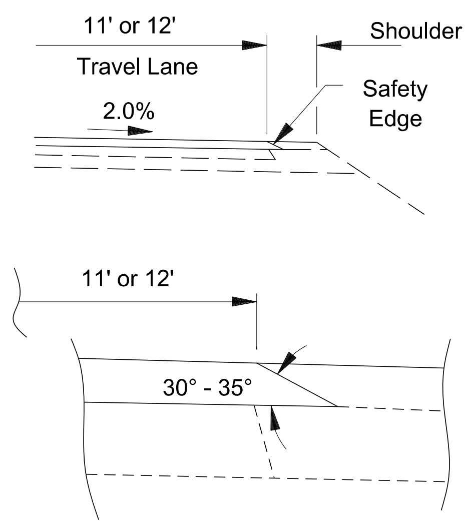
(a) New Construction



(b) Overlay 14' Pavement



(c) Overlay
11' or 12' Pavement
with Trench Widening



(d) Overlay
11' or 12' Pavement
without Trench Widening

SAFETY EDGE DETAILS
Figure 13-2-C

13-2.03.06 Horizontal Limits for Chemical Treatments

- Treated Granular Material – The horizontal limits for the chemical treatment of granular material should be shown two feet outside the pavement edges of the roadway. For example, on a roadway with two 12-foot lanes and a total pavement width of 28 feet (including two-foot paved shoulders), a width of 32 feet should be shown for the chemical treatment of granular material.

- Treated Subgrade – The horizontal limits for chemical treatment of the subgrade should be shown from the outside edge of shoulder to the outside edge of shoulder.

13-2.04 Bridge End Pavement

Due to potential differential settlement between a bridge deck and adjacent roadway pavement, a reinforced concrete slab, referred to as the bridge end pavement, which is the same width as the paving bracket on the bridge, should be provided to ensure a continuing smooth transition. Typically, a separate typical section is provided in the set of plans. Other design details are included in the *Standard Drawings.*

13-2.05 District Pavement Structure Recommendation

The District should submit the pavement structure recommendation to Roadway Design Division prior to the beginning of the development of Contract Plans. This recommendation should include the following information:

1. results from the final soil profile and earthwork recommendation and the resulting design California Bearing Ratio (CBR)

2. pavement structure for all new pavements to be placed on the project, which may also include supplemental or temporary roadways

3. type of treatment of subgrade (including estimated lengths or breakdown by percentage of project length if multiple types are recommended)

4. shoulder design and thickness

5. mix designation type for each lift

6. computation of structure numbers for all sections of new pavements

7. rehabilitation strategy for existing pavement that does not warrant full replacement

13-3.0 ESTIMATING QUANTITIES

Computation worksheets should be documented and placed in the project file. The estimated quantities and pay item numbers should be recorded on the Summary of Quantities sheets in the plans.

Generally, the quantity estimates should be determined according to the following criteria:

1. Concrete Pavement – Concrete pavement is measured in square yards of surface area. This pay item should include the proposed thickness.

2. Asphalt Pavement – Asphalt pavement is estimated in tons of material. A unit weight of 94 pounds per square yard per inch of pavement thickness may be assumed for OGFC for estimating purposes. A unit weight of 110 pounds per square yard per inch of pavement thickness may be assumed for all other mix designs. Separate estimates should be provided for each of the identified pay items for the proposed asphalt pavement.

3. Prime Coat – Bituminous prime coat should not be called out on the typical sections if the granular material is cement-treated. A bituminous prime coat may be specified where pavement is to be placed on an untreated granular base course. The prime coat should be expressed in gallons at a rate of application of 0.35 gallons per square yard. The width of the prime coat should typically be two feet greater than the pavement width (one foot on each side).

4. Granular Material and Crushed Stone – Granular material and crushed stone should be measured and estimated by the cubic yard or ton. Each different class or group should be identified and designated separately.

   When measured by the ton, the weight should be determined by an estimated unit dry weight for each class as follows:

Table 13-3-A
GRANULAR MATERIAL UNIT WEIGHTS AND SHRINKAGE FACTORS

Material	Pounds per cubic foot (CR)	Tons per cubic yard
Granular Class 1 and 2	131	1.77
Granular Class 3 and 4	129	1.74
Granular Class 5 and 6	127	1.71
Crushed Stone	125	1.69
Soil	100	1.35
Granular (Other Classes)	114	1.54

   The following shrinkage factors should be used for granular material and crushed stone:

Pay Item Description	Shrinkage Factor
LVM (cubic yards)	50% (multiply by 1.5)
AEA (cubic yards)	0% (no shrinkage)
Ton (ton)	20% (multiply by 1.2)

   The percentage increase should be noted in a footnote on the Summary of Quantities sheet when the quantity includes either the 50% or 20% increase.

5. Cement-Treated Base Course – The following pay items should be estimated for cement-treated courses:

- cubic yards of granular material
- tons of Portland cement
- square yards of soil-cement-water mixing (for each depth course)

An estimated rate of cement application (R) is typically provided, which should be used as a base rate for plan quantities for cement in terms of dry weight granular material.

Soil-cement-water mixing (WM) is measured by the square yard.

The following equation should be used to estimate Portland cement:

$$PC = (L)(W)(D)(CR)(1/2000)(R)(0.01) \quad \text{(Equation 13-3-1)}$$
$$= (L)(W)(D)(CR)(R)(1/200{,}000)$$

$$WM = (L)(W)(1/9)$$

Where:

PC	=	Portland cement (tons)
WM	=	Soil cement-water mixing (square yards)
L	=	Length (feet)
W	=	Width of treatment (feet)
D	=	Depth of treatment (feet)
CR	=	Conversion rate for granular material (pounds per cubic foot) (See Table 13-3-A)
1/2000	=	Factor to convert pounds to tons
R	=	Rate of cement application by weight (e.g., for 4%, R=4)
0.01	=	Factor to convert to decimal (%)

6. Lime-Treated Subgrade – The following contract pay items should be estimated for lime-treated subgrade:

- tons of hydrated lime
- square yards of soil-lime-water mixing (i.e., for each depth and class)

Soil-lime-water mixing should be measured by the square yard for each course. Only one mixing operation is estimated regardless of the number of applications of materials and the actual number of mixing operations.

The following equations should be used to estimate hydrated lime:

$$HL = (L)(W)(D)(100)(1/2000)(R)(0.01) \quad \text{(Equation 13-3-2)}$$
$$= (1/2000)(L)(W)(D)(R)$$

$$WM = (L)(W)(1/9)$$

Where:

HL	=	Hydrated lime (tons)

WM = Soil lime water mixing (square yards)
L = Length (feet)
W = Width of treatment (feet)
D = Depth (feet)
100 = Estimated weight of soil in pounds per cubic foot
1/2000 = Factor to convert pounds to tons
R = Percent (%)
0.01 = Factor to convert to decimal (%)

7. Cement-Treated Subgrade – The following contract pay items should be estimated for cement-treated subgrade:

- tons of Portland cement
- square yards of soil-cement-water mixing (i.e., for each depth course)

The rate of application for the cement varies depending upon the soils analysis for the individual project. The station limits and width for treatment in terms of percent of dry weight is normally determined during construction.

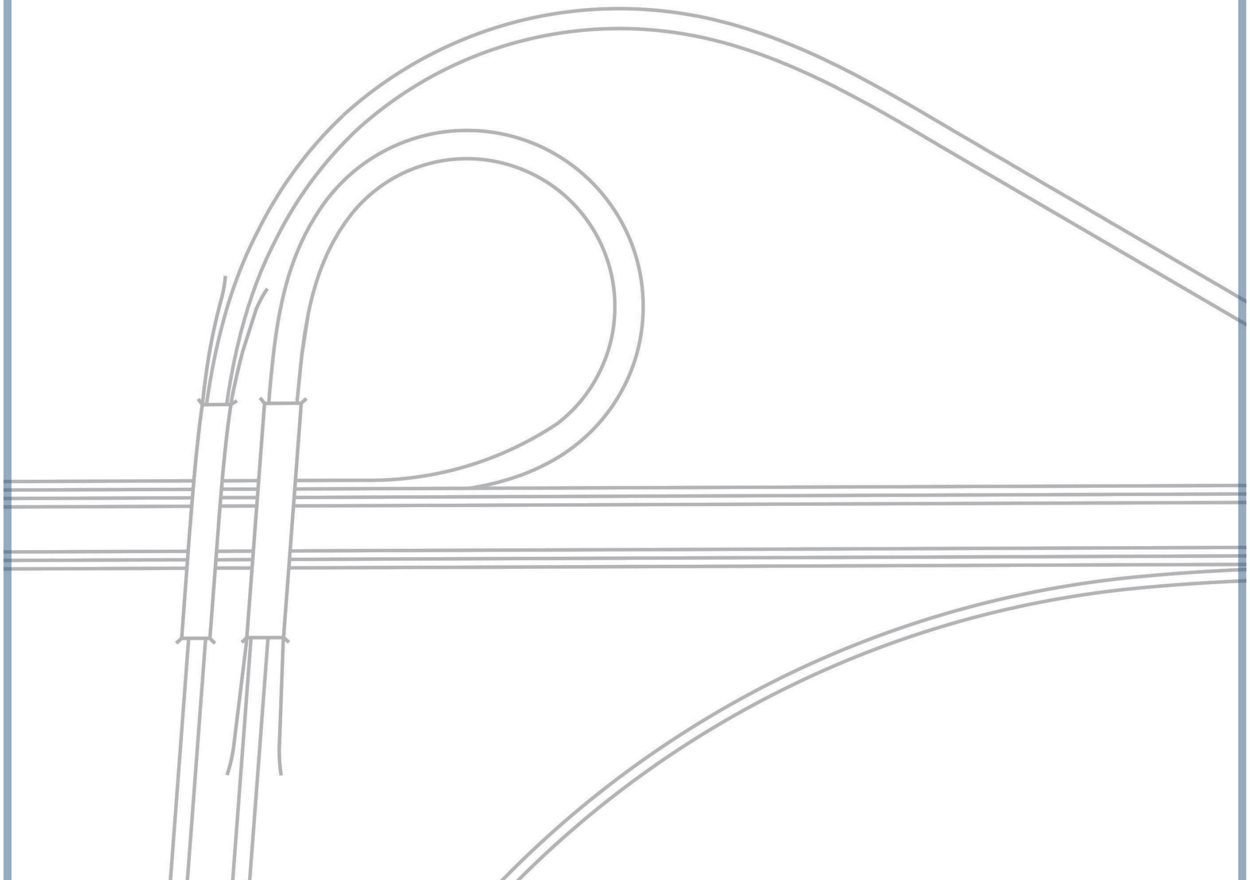
Soil-cement-water mixing is measured by the square yard.

The following equation should be used to estimate Portland cement:

$$PC = (L)(W)(D)(CR)(1/2000)(R)(0.01) \quad \text{(Equation 13-3-3)}$$
$$= (L)(W)(D)(CR)(R)(1/200{,}000)$$

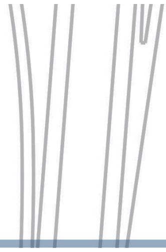
$$WM = (L)(W)(1/9)$$

Where:
PC = Portland cement (tons)
WM = Soil cement-water mixing (square yards)
L = Length (feet)
W = Width of treatment (feet)
D = Depth of treatment (feet)
CR = Conversion rate for granular material (pounds per cubic foot) (See Table 13-3-A)
1/2000 = Factor to convert pounds to tons
R = Percent of cement application (e.g., for 4%, R=4)
0.01 = Factor to convert to decimal (%)



CHAPTER 14

Geometric Design of Urban Roadways



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Chapter 14
GEOMETRIC DESIGN OF URBAN ROADWAYS

Because rural and urban areas vary in the nature and intensity of development and in their relative traffic volumes, different geometric design criteria are warranted for urban roadways relative to the criteria for rural roadways. Differences include, but are not limited to, the criteria for design speed, superelevation rate, length of vertical curve, and the roadside clear zone. In addition, the need for parking lanes, sidewalks, bicycle lanes, and the use of curbed sections in urban areas should be evaluated.

This chapter provides the geometric design criteria applicable to new construction/reconstruction projects on urban roadways. However, in many cases, the criteria and procedures presented in other chapters are equally applicable to urban roadways.

14-1.0 BASIC DESIGN CONTROLS

14-1.01 Functional Classification System

The Department's design criteria for urban roadways are based on the functional classification system. Classifications for urbanized areas include arterial, collectors, and local streets. The functional classification designations lead directly to the criteria provided in the geometric design criteria tables in Section 14-2.08. Section 2-1.01 discusses the functional classification system.

14-1.02 Project Scope of Work

The project scope of work may be new construction, reconstruction, 3R, preventive maintenance, or spot improvement. The scope of work definitions presented in Section 2-7.0 also apply to projects on urban roadways.

14-2.0 URBAN ROADWAYS

The criteria in this section apply to all urban functional classes unless otherwise noted.

14-2.01 Design Traffic Volume

14-2.01.01 Design Hourly Volume

For new construction/reconstruction projects, urban roadways should be designed to accommodate traffic volumes that may occur within the projected life of the facility (usually 20 years from completion of construction). The Department's Planning Division maintains current records of traffic data, including the design hourly volume (DHV) and average daily traffic (ADT).

14-2.01.02 Level of Service

The roadway mainline and at-grade intersections should be designed to accommodate the DHV at the Level of Service (LOS) threshold. The Planning Division should be consulted if a capacity analysis is needed for a specific project. Section 2-3.01 provides roadway capacity definitions. Levels of service criteria for urban roadways are provided in the geometric design criteria tables in Section 14-2.08.

14-2.02 Design Speed

A design speed should be selected for urban roadways based on the adjacent land use and the functional classification using the geometric design criteria tables in Section 14-2.08. Geometric design elements should be consistent with the selected design speed. Design speed is discussed in more detail in Section 2-2.01.

14-2.03 Stopping Sight Distance

Stopping Sight Distances (SSD) for various design speeds are provided in the geometric design criteria tables in Section 14-2.08. SSD is measured from the driver's eye height 3.5 feet above the pavement to a 2-foot object height. Sight distance is discussed in more detail in Section 2-8.0.

14-2.04 Horizontal Alignment

The three sets of criteria for horizontal alignment, which are based on design speed, are summarized as follows:

- V > 50 miles per hour – The criteria and procedures presented in Chapter 3, "Horizontal Alignment", apply to urban roadways with design speeds greater than 50 miles per hour, including e_{max} = 10% and a theoretical distribution of "e" and "f" that AASHTO has designated as Method 5.

- V = 50 miles per hour – For urban roadways with a design speed of 50 miles per hour, e_{max} = 6% using the AASHTO Method 5 distribution between "e" and "f." The *Standard Drawings* present the details of superelevation transitions for these facilities.

- V ≤ 45 miles per hour – For urban roadways with a design speed less than or equal to 45 miles per hour, e_{max} = 4%. The distribution of "e" and "f" is based on the AASHTO Method 2.

See AASHTO's *A Policy on Geometric Design of Highways and Streets* for more information on distribution methods.

14-2.04.01 Curve Types

Deflection changes in alignment on low-speed urban roadways (V ≤ 45 miles per hour) are usually accomplished by using a simple curve. Where some physical control prevents the use of a simple curve, a compound curve may be used. Section 3-3.01 discusses the details of simple and compound curves.

Very small deflection angles may not require a horizontal curve; i.e., the roadway may be designed with an angular break. As a general guide, the need for a curve when the deflection angle is less than one degree should be evaluated, considering roadside development, construction costs, and the visibility of the kink if left in place.

14-2.04.02 Minimum Radii and Superelevation Rate

The minimum radii for various design speeds and functional classification are provided in the geometric design criteria tables in Section 14-2.08. A larger radius should be provided where feasible, especially for projects on new location, in order to avoid future sight obstructions as the area develops. See Section 14-2.04.05 and Section 3-5.0 for information on sight obstructions on the inside of horizontal curves.

The Standard Drawings provide applicable radii for various combinations of superelevation rates for each design speed. The indicated superelevation rates are applicable regardless of the number of lanes. Table 14-2-B (e_{max} = 4%) and Table 14-2-C (e_{max} = 6%) provide the minimum radii for a normal crown slope and a reverse crown slope at various design speeds for new construction.

The scope of work for some projects on existing low-speed urban roadways ($V \leq 45$ miles per hour) may be such that additional capacity is being provided, and therefore such projects would be subject to new construction/reconstruction criteria. In such cases, superelevation on horizontal curves may not be feasible due to, but not limited to, the following factors:

1. right of way and utility impacts
2. historical property impacts
3. surface drainage considerations
4. frequency of intersecting roadways and driveways
5. pedestrian facility impacts

Therefore, the existing cross slope may be retained unless a crash pattern exists that is related to the horizontal curve. In such cases, consideration should be given to providing cross slopes that meet the values provided in Table 14-2-A.

Table 14-2-A
MINIMUM RADII AND SUPERELEVATION FOR LOW-SPEED URBAN ROADWAYS
(V ≤ 45 MILES PER HOUR)

e (%)	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph
	Radius (ft)	Radius (ft)	Radius (ft)	Radius (ft)	Radius (ft)	Radius (ft)
-2.0%	107 ≤ R < 108	198 ≤ R < 200	333 ≤ R < 337	510 ≤ R < 517	762 ≤ R < 773	1039 ≤ R < 1055
-1.5%	105 ≤ R < 107	194 ≤ R < 198	324 ≤ R < 333	495 ≤ R < 510	736 ≤ R < 762	1000 ≤ R < 1039
0%	99 ≤ R < 105	181 ≤ R < 194	300 ≤ R < 324	454 ≤ R < 495	667 ≤ R < 736	900 ≤ R < 1000
1.5%	94 ≤ R < 99	170 ≤ R < 181	279 ≤ R < 300	419 ≤ R < 454	610 ≤ R < 667	818 ≤ R < 900
2.0%	92 ≤ R < 94	167 ≤ R < 170	273 ≤ R < 279	408 ≤ R < 419	593 ≤ R < 610	794 ≤ R < 818
2.2%	91 ≤ R < 92	165 ≤ R < 167	270 ≤ R < 273	404 ≤ R < 408	586 ≤ R < 593	785 ≤ R < 794
2.4%	R = 91	164 ≤ R < 165	268 ≤ R < 270	400 ≤ R < 404	580 ≤ R < 586	776 ≤ R < 785
2.6%	90 ≤ R < 91	163 ≤ R < 164	265 ≤ R < 268	396 ≤ R < 400	573 ≤ R < 580	767 ≤ R < 776
2.8%	89 ≤ R < 90	161 ≤ R < 163	263 ≤ R < 265	393 ≤ R < 396	567 ≤ R < 573	758 ≤ R < 767
3.0%	R = 89	160 ≤ R < 161	261 ≤ R < 263	389 ≤ R < 393	561 ≤ R < 567	750 ≤ R < 758
3.2%	88 ≤ R < 89	159 ≤ R < 160	259 ≤ R < 261	385 ≤ R < 389	556 ≤ R < 561	742 ≤ R < 750
3.4%	R = 88	158 ≤ R < 159	256 ≤ R < 259	382 ≤ R < 385	550 ≤ R < 556	734 ≤ R < 742
3.6%	87 ≤ R < 88	157 ≤ R < 158	254 ≤ R < 256	378 ≤ R < 382	544 ≤ R < 550	726 ≤ R < 734
3.8%	R = 87	155 ≤ R < 157	252 ≤ R < 254	375 ≤ R < 378	539 ≤ R < 544	718 ≤ R < 726
4.0%	86 ≤ R < 87	154 ≤ R < 155	250 ≤ R < 252	371 ≤ R < 375	533 ≤ R < 539	711 ≤ R < 718

Notes:

1. *Computed using Superelevation Distribution Method 2 as shown in AASHTO's* A Policy on Geometric Design of Highways and Streets.
2. *Superelevation may be optional on low-speed urban roadways.*

$$R = \frac{V_d^2}{15\,(e_d + f)}$$

Where:

R = *radius of curve, feet*
e_d = *design superelevation rate*
f = *side-friction factor*
V_d = *design speed, miles per hour*

Table 14-2-B
MINIMUM RADII FOR NORMAL CROWN AND REVERSE CROWN (2% Typical)

Design Speed (mph)	Minimum Radius (ft)		
	Normal Crown	Reverse Crown	See *Standard Drawings*
30	R ≥ 2,830	2,830 > R ≥ 1,880	R < 1,880
35	R ≥ 3,730	3,730 > R ≥ 2,490	R < 2,490
40	R ≥ 4,770	4,770 > R ≥ 3,220	R < 3,220
45	R ≥ 5,930	5,930 > R ≥ 4,040	R < 4,040
50	R ≥ 7,220	7,220 > R ≥ 4,940	R < 4,940

Note: e_{max} *= 4%*

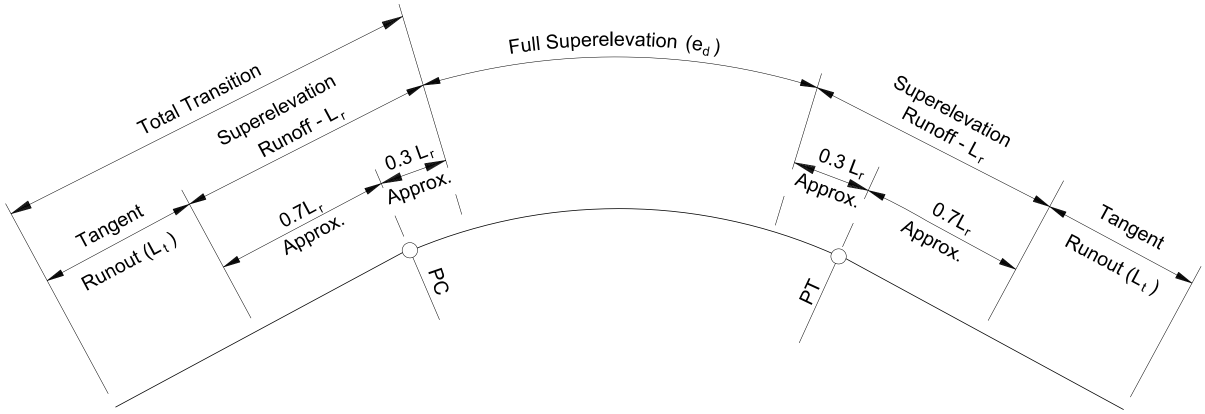
Table 14-2-C
MINIMUM RADII FOR NORMAL CROWN AND REVERSE CROWN (2% Typical)

Design Speed (mph)	Minimum Radius (ft)		
	Normal Crown	Reverse Crown	See *Standard Drawings*
30	R ≥ 3,130	3,130 > R ≥ 2,240	R < 2,240
35	R ≥ 4,100	4,100 > R ≥ 2,950	R < 2,950
40	R ≥ 5,230	5,230 > R ≥ 3,770	R < 3,770
45	R ≥ 6,480	6,480 > R ≥ 4,680	R < 4,680
50	R ≥ 7,870	7,870 > R ≥ 5,700	R < 5,700

Note: e_{max} *= 6%*

14-2.04.03 Superelevation Transition Length

The total superelevation transition length consists of two segments — the length of tangent runout (L_t) and the length of superelevation runoff (L_r). Figure 14-2-A illustrates the typical applications to urban roadways for all design speeds. Table 3-4-C provides the maximum relative gradients for superelevation transition lengths. Superelevation transition length is discussed in detail in Section 3-4.02.



Note: Superelevation runoff and runout lengths should be as shown in the Standard Drawings*.*

APPLICATION OF THE SUPERELEVATION TRANSITION LENGTH
Figure 14-2-A

14-2.04.04 Superelevation Axis of Rotation

The superelevation axis of rotation on undivided urban roadways most often follows the centerline of the roadway, which applies to 2-lane, 3-lane, 4-lane, and 5-lane sections. However, the pavement may be rotated about the outside edge of the traveled way on undivided roadways, where factors such as roadside development, drainage, or utilities make it preferable to keep one edge at a fixed location. Superelevation axis of rotation is discussed in more detail in Section 3-4.03.

On divided urban roadways with a raised median, the axes of rotation are typically about the two median edges, as discussed in Section 3-4.03.02. However, each of the divided roadways may be superelevated such that each roadway is sloping away from the raised median, thereby avoiding the need for drainage accommodations (e.g., inlets) in the median. In such cases, the following additional information should be applied:

- Any horizontal curves should not exceed the minimum radii for normal crown cross slope so that the adverse cross slope for the driver on the outside of a horizontal curve is limited to two percent.

- The superelevation of left-turn lanes should be adjusted to minimize the effect of a warped median without exceeding the maximum break-over slope between the turn lane and the through lane.

If the shoulders have curb and gutter, the gutter slope direction should be designed to match the shoulder cross slope direction. The *Standard Drawings* should be referenced for additional details on superelevation development.

14-2.04.05 Horizontal Sight Line Offset

The horizontal sight line offset (HSO) criteria presented in Section 3-5.0 apply to urban roadways. A larger radius should be provided where feasible, especially where the minimum radius will not provide the necessary HSO for SSD around horizontal curves. Section 3-5.0 should be referenced for additional information on the HSO.

14-2.05 Vertical Alignment

The criteria and procedures presented in Chapter 4, “Vertical Alignment”, apply to urban roadways, with additional information provided in the following sections.

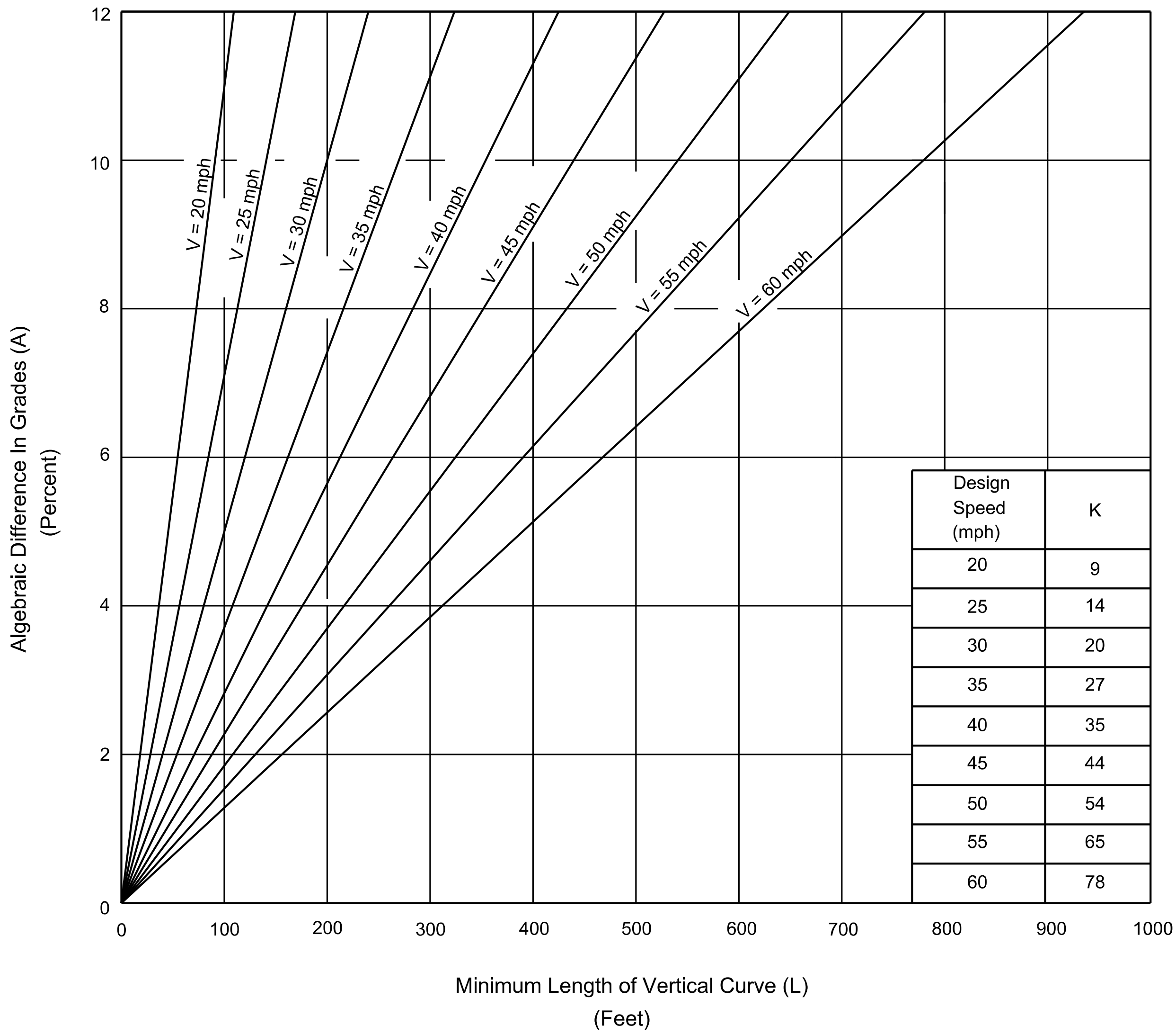
14-2.05.01 Grades

Grades on urban projects should be based on the functional classification and on controlling conditions, including existing roadways, type of terrain (i.e., level or rolling), and adjacent property development. Maximum grades for urban roadways are provided in the geometric design criteria tables in Section 14-2.08. Where outside controlling factors are not severe, the normal practice of carrying the plan grade on the centerline or on the median edges is acceptable. Where the controls are significant, the main profile may be supplemented with other elevation controls (e.g., gutter-line profiles, top-of-curb profiles). Supplemental controls should be shown on the typical section(s) and on the plan-profile sheets.

Proposed grades should not include flat areas where water would pond. Curb lines should have a minimum gradient of 0.2%, but preferably should be 0.4%. However, in certain areas of the state, such as the Mississippi Delta and the coast, the terrain may be so flat that providing a minimum gradient would not be feasible without incorporating an undesirable rolling profile. Engineering judgment should be used in such cases. Curb elevations should be designed to permit drainage flow into the gutter and to avoid ponding of water behind the curb. The high point of crest vertical curves near intersecting roadways should desirably be located near the centerline of intersecting cross streets. See Section 6-7.0 for additional information.

14-2.05.02 Vertical Curves

Where vertical curves are proposed, they should be long enough to provide the recommended SSD. The criteria in Section 4-5.0 also apply to urban roadways. In lighted areas on urban roadways, the comfort criteria for the design of sag vertical curves may be used. Figure 14-2-B provides the lengths of sag vertical curves for various design speeds. The minimum length of vertical curves (crest and sag) for urban roadways should be 50 feet. Where the algebraic difference in grades is 0.2% or less on urban roadways, vertical curves may be omitted.



Design Speed (mph)	K
20	9
25	14
30	20
35	27
40	35
45	44
50	54
55	65
60	78

$$L = \frac{AV^2}{46.5}$$

$$L = KA$$

Where:

L	=	*Length of vertical curve, feet*
A	=	*Algebraic difference between grades, %*
K	=	*Horizontal distance to cause a 1% change in gradient, feet*

Notes:

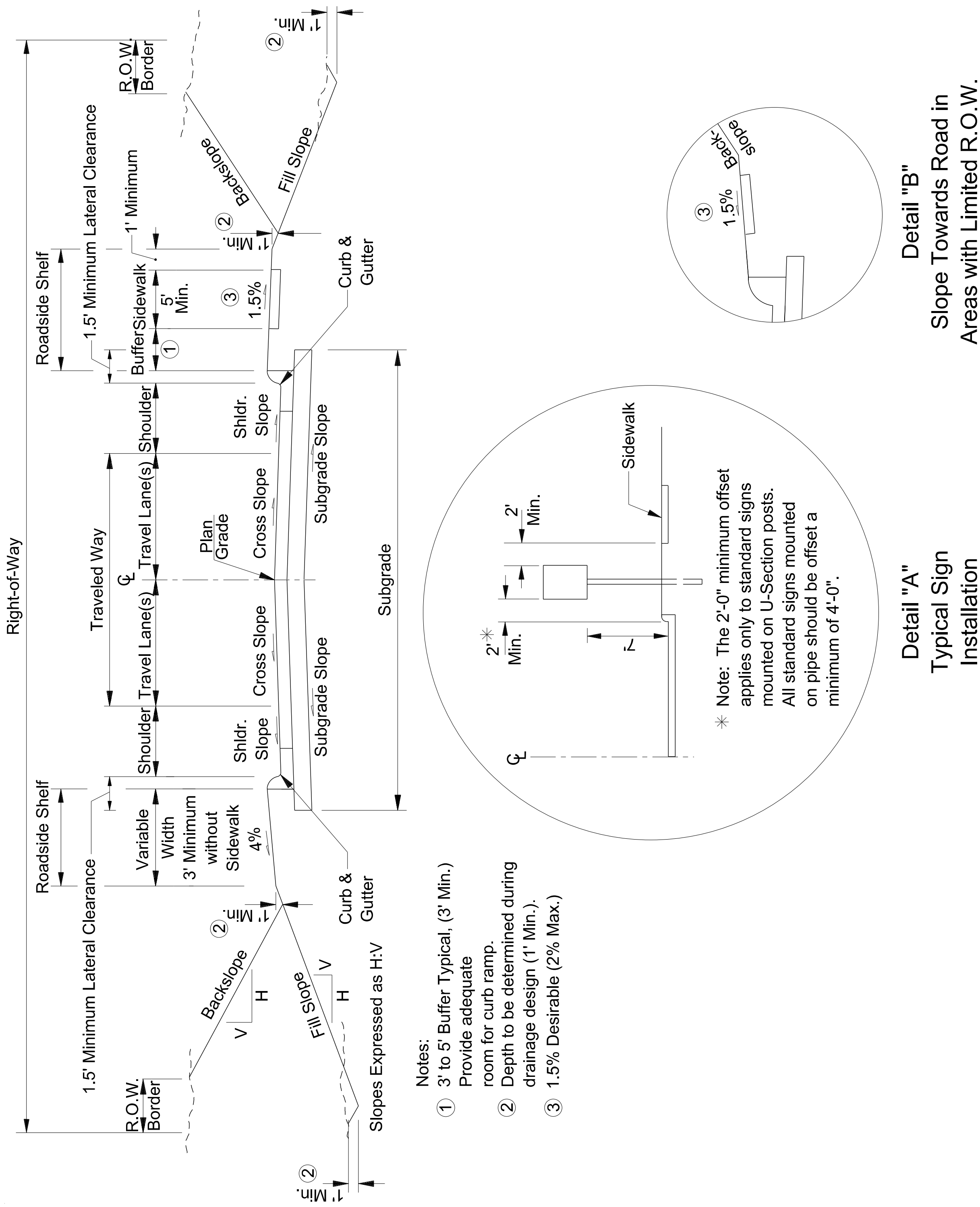
1. L_{min} *= 50 feet*
2. *This figure only applies in lighted areas on new construction and reconstruction. For non-lighted areas, the criteria in Section 4-5.0 should be used.*

LENGTH OF SAG VERTICAL CURVES
(Comfort Criteria, Lighted Areas Only)
Figure 14-2-B

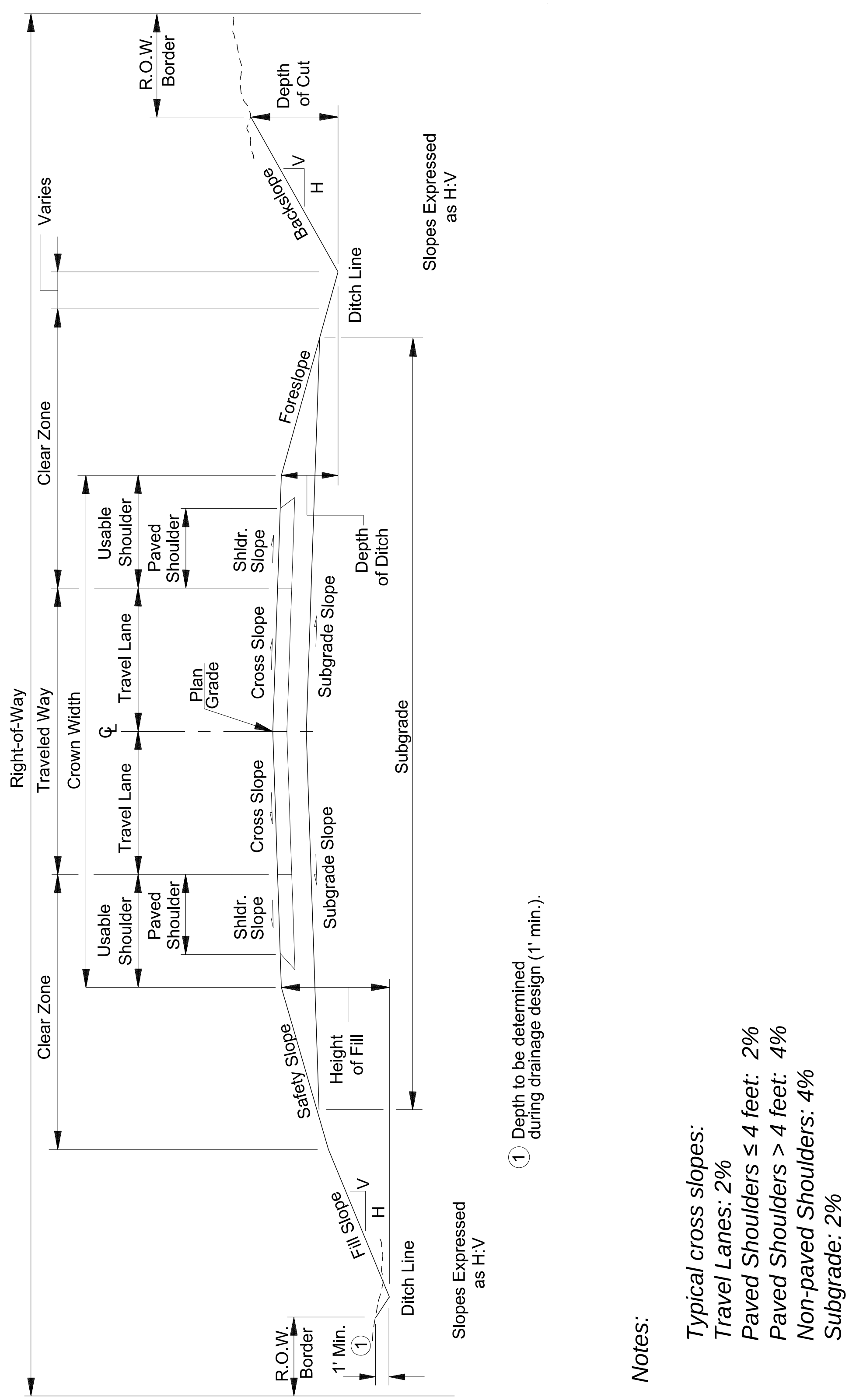
14-2.06 Cross-Section Elements

Figures 14-2-C and 14-2-D illustrate typical roadway cross sections for urban roadways with and without curbs. The primary purpose of these figures is to describe the nomenclature used in urban roadway design; however, the figures are not intended to illustrate every aspect of the desirable typical section. Every project is unique and may include additional features not depicted in the figures (e.g., inlets, bicycle lanes, shared-use paths, on-street parking).

The design criteria for cross-section elements are provided in the geometric design criteria tables in Section 14-2.08. Section 2-8.03 presents a discussion on each cross-section element, which applies in part to the design of urban roadways. The following subsections should be reviewed for additional information on urban roadway projects.



TYPICAL SECTION NOMENCLATURE (Urban Roadways with Curbs)
Figure 14-2-C



TYPICAL SECTION NOMENCLATURE (Urban Roadways without Curbs)
Figure 14-2-D

14-2.06.01 Curb and Gutter Usage

Curbs are often provided on urban roadways to control drainage, provide separation between vehicles and pedestrians, and control access. The decision to use a curb should be based on an evaluation of benefits, installation costs, and right of way impacts. Curbs should not be used on freeways because an out-of-control vehicle may overturn or become airborne after an impact with the curb.

The two classes of curbs are vertical curbs and sloping curbs. Both curb types typically include a gutter pan to form a combination curb and gutter section. Details of the different types of curb and gutter used by the Department are provided in the *Standard Drawings*. The following additional information should be applied when curb and gutter is proposed on urban non-freeway roadways:

1. Vertical Curbs – Vertical curbs have a relatively steep face and are intended to help prevent out-of-control vehicles from leaving the roadway. The *Standard Drawings* include details of a vertical curb known as Type 1 curb and gutter, which has a height of six inches. This type of curb and gutter is preferred where sidewalks are proposed, especially if the full shoulder width and/or the minimum buffer width is not provided. However, Type 1 curb and gutter should only be used on low-speed roadways ($V \leq 45$ miles per hour).

2. Sloping Curbs – A sloping curb should be used where curb and gutter is proposed on high-speed roadways ($V \geq 50$ miles per hour). The *Standard Drawings* include two types of curb and gutter with sloping curb faces — Type 2 and Type 3. The following information should be applied when determining which type of sloping curb to use:

   a. Type 2 – Type 2 curb and gutter has a curb face that is on a plane slope that reaches a height of four inches at the back of the curb. Typical locations include along the radii of at-grade intersections and the edges of channelized islands. Type 2 curb and gutter should also be used along high-speed urban roadways with minimal shoulder widths where sidewalks are not included, such as along the edges of raised medians. Using Type 2 curb gutter in this situation may help out-of-control vehicles from being redirected into the adjacent travel lane.

   b. Type 3 – Type 3 curb and gutter has a round-faced curb with a height of six inches. Typical locations include along the pavement edges of high-speed urban roadways when open shoulders are not feasible. Shoulder widths wider than the minimum width should be provided, especially where sidewalks are included, to minimize the possibility of out-of-control vehicles being redirected into the adjacent travel lane or becoming airborne after an impact with the curb.

3. Curb Placement – The curb face along the roadway should be offset at the outside edge of the shoulder or a minimum of two feet from the traveled way, whichever is greater. Sections 14-2.06.02 and 14-2.06.03 should be reviewed for additional information regarding the placement of curb and gutter relative to travel lane and shoulder width. Section 14-2.06.05 should be referenced for additional information regarding the placement of curb and gutter relative to roadways with sidewalks. Section 6-2.04 should be referenced for information on curb and gutter usage at intersections.

4. Drainage – Where inlets are necessary to facilitate storm drainage, the type of inlet used is dependent upon the type of curb and gutter that is proposed. Where proposed curb and gutter is not relative to the discussion in Item 2, the type of curb and gutter chosen may then be based upon the preference between curb inlets and gutter inlets.

5. Curb/Guardrail Combinations - Curbs should desirably not be used in conjunction with guardrail. Where curb and guardrail are used together, the face of the curb should be flush with the face of the guardrail, or the guardrail should protrude out beyond the face of the curb, to ensure that the curb does not have any effect on how a vehicle strikes the guardrail. Chapter 9, “Roadside Safety”, should be referenced for additional information.

14-2.06.02 Lane Widths

Paved lane widths vary according to functional classification. Minimum lane widths are provided in Section 14-2.08. Where a curb and gutter section is provided, the gutter pan should not be used as part of the traveled way, but may be used as part of the shoulder width.

14-2.06.03 Shoulder Widths

The geometric design criteria tables in Section 14-2.08 provide the minimum shoulder widths for the various functional classifications of urban roadways. In some cases, desirable shoulder widths are also provided. Where feasible, the desirable widths should be provided to allow space to maneuver and for emergency parking. However, space for the shoulder width may be limited by available right of way in developed areas. The following information should be considered to determine shoulder widths on urban roadways:

1. On curbed roadways, the shoulder widths are measured from the edge of the outside travel lane to the face of the curb. See Section 14-2.06.01 for additional information on shoulder widths relative to the different types of curb and gutter that may be used on urban roadways.

2. On curbed roadways, the shoulder width should be based on a balance between several factors, including but not limited to:

   a. minimizing the probability of being struck by an out-of-control vehicle
   b. providing sufficient buffer between the traveled way and sidewalks
   c. minimizing the amount of ponding of surface drainage in the traveled way

3. Right of way impacts, design speed, traffic volumes, and the potential use of a wider shoulder as a continuous right-turn lane should be evaluated (particularly where frequent points of access to the roadway exist) when selecting the shoulder width on curbed roadways.

4. Section 14-2.06.06 provides criteria for the design of on-street parking. However, if the parking lane may be converted to a travel lane in the future, consideration should be given to increasing the width accordingly.

5. Where curbs are provided with shoulders wider than two feet, paved shoulders are required between the gutter pan and edge of traveled way.

6. Where adequate width is provided, paved shoulders can be designated as bicycle lanes with the necessary pavement markings. See Section 8-6.04 for the design of bicycle lanes. The *AASHTO Guide for the Development of Bicycle Facilities* should be referenced for additional information.

14-2.06.04 Roadside Shelf

The roadside shelf is the portion of the roadway from the back of curb to the top of the "V" ditch or fill slope, as shown in Figure 14-2-C. The following information should be considered:

1. Fill Sections – The roadside shelf should slope away from the curb, as shown in Figure 14-2-C, to reduce the amount of runoff entering the storm drainage system. A fill ditch should be provided to carry the side drainage to the next cross drain.

2. Cut Sections – Desirably, the roadside shelf should slope away from the curb with a ditch behind the sidewalk. However, it may not be feasible to slope the roadside shelf away from the curb in urban areas. Sloping the roadside shelf towards the roadway, as shown in Detail B of Figure 14-2-C, may be an alternative. In such cases, the storm drainage system should be designed to accommodate the additional runoff. The decision on which way to slope the roadside shelf should be determined on a case-by-case basis.

3. Alternating Fill and Cut Sections – In locations where it is not feasible to provide the designs discussed in Items 1 and 2 above, and where alternating fill and cut sections would result in alternating directions of the slope of the roadside shelf behind the curb, it is acceptable to slope this area towards the roadway as shown in Detail B, even in fill sections. The storm drainage system should be designed to accommodate the additional runoff.

4. Without Sidewalks – The desirable roadside shelf width is five feet, sloping at 4% away from the roadway. However, the minimum width for the roadside shelf should be three feet where right of way and/or environmental constraints exist.

5. Ditch Depth – Ditches should be designed to accommodate the proposed drainage. At a minimum, the ditch depth should be one foot.

14-2.06.05 Sidewalks

Sidewalks are integral parts of the urban environment and should be considered in the design of all projects on urban roadways. Each municipality has an *ADA* transition plan that should be referenced to ensure that the scope of the project is consistent with the needs of pedestrians. Documentation should be included in the project file when any features of the *ADA* transition plan are not included in the scope of the project.

14-2.06.05.1 *Accessibility for Disabled Individuals*

A pedestrian access route shall be provided within sidewalks and other pedestrian circulation paths, pedestrian street crossings, and pedestrian overpasses and underpasses. A pedestrian access route is a continuous and unobstructed path of travel provided for pedestrians with disabilities within or coinciding with a pedestrian circulation path in the public right of way.

Pedestrian facilities shall meet the Department's accessibility criteria, which comply with the current United States Access Board publication *Public Right-of-Way Accessibility Guidelines* (*PROWAG*). These criteria were developed for new construction projects, but many projects in the public right of way involve improvements to existing roadways that are subject to new construction/reconstruction criteria. Any pedestrian access elements that are altered by the project shall be reconstructed to comply with the applicable requirements for new construction. However, some flexibility is allowed for projects on existing roadways where physical constraints make it infeasible to fully comply with the applicable requirements. Existing physical constraints include, but are not limited to:

1. Right of way availability
2. Utilities
3. Drainage structures
4. Historic features

In such cases, the pedestrian access elements shall be reconstructed to comply with the applicable requirements to the maximum extent feasible within the scope of the project. See Chapter 12, "Existing Roadways", for more information about projects on existing roadways.

14-2.06.05.2 *Existing Roadways*

The following information applies to the placement or replacement of sidewalks on existing roadways:

1. <u>Sidewalks Currently Exist (Roadway and Bridge)</u> – Where sidewalks currently exist along a roadway, non-compliant sidewalks that are affected by a project shall be reconstructed to meet *ADA* requirements to the maximum extent feasible within the scope of the project. If a bridge with an existing sidewalk is replaced, the sidewalk shall also be replaced.

2. <u>Bridge without Sidewalk/Roadway with Sidewalk</u> – If a bridge without a sidewalk will be replaced and if existing sidewalks approach the bridge, the sidewalks shall be extended across the bridge.

3. <u>One Side versus Two Sides</u> – In urban areas, sidewalks on both sides of the roadway are preferred. Sidewalks on only one side of the roadway reduce connectivity of the pedestrian network, as well as pedestrian safety and accessibility. If a sidewalk only exists on one side of the roadway or bridge, the criteria in Item 4 below should be reviewed to determine if the construction of a new sidewalk on the other side is warranted.

4. <u>Sidewalks Currently Do Not Exist</u> – The need for sidewalks should be determined on a case-by-case basis. The following guidelines should be considered:

    a. Consideration should be given to providing sidewalks and/or curb ramps along any roadway or bridge where pedestrian traffic currently exists or is expected.

    b. Sidewalks should be considered whenever the roadway drainage is changed from open shoulders to a curb-and-gutter section.

    c. Sidewalks should be considered at points of community development that result in pedestrian concentrations along the roadway. Roadways leading to schools or libraries should have accessible sidewalks on at least one side.

d. The justification for sidewalks depends in part on the vehicle/pedestrian hazard, which is governed by the volume of pedestrians, as well as the volume and speed of vehicular traffic.

e. In conducting the evaluation for providing sidewalks, the constraints of adding a sidewalk (e.g., right of way, drainage, utility relocations, environmental impacts, structural impacts on bridges) should be considered. When a decision is made not to provide a sidewalk due to constraints, the justification should be documented for reference.

f. Public involvement is encouraged when deciding whether a sidewalk should be considered on a project where pedestrian traffic is expected.

14-2.06.05.3 *Sidewalk Design Criteria*

In determining the sidewalk design, the following information should be applied:

1. Safety – Design features of the sidewalk should allow pedestrians to have a sense of security and predictability. Sidewalk users should not feel they are at risk due to the presence of adjacent traffic.

2. Continuity – Walking routes should be obvious to all pedestrians and should not require them to travel out of their way unnecessarily.

3. Sidewalk Width – The minimum width of sidewalks is five feet. However, in Central Business District (CBD) areas, wider sidewalks may be warranted. Where roadside appurtenances (e.g., utility poles, traffic signs, fire hydrants, benches, planters) exist or are proposed, a clear and unobstructed path shall be provided that is no less than four feet in width.

4. Buffer Area With Curb – Buffer areas between the curb and sidewalk should be provided unless there are constraints (e.g. right of way, utilities). On-street parking can serve as an acceptable buffer, but where no on-street parking exists, the minimum buffer width between the edge of pavement and the edge of sidewalk should be three feet; however, wider buffer areas are desirable as they help facilitate vertical transitions at driveway entrances. Where there are such constraints, the buffer area may be omitted; however, stronger consideration should be given to providing the minimum buffer width on high-speed roadways ($V \geq 50$ miles per hour) or roadways with heavy traffic volumes, if feasible. If the buffer width cannot be provided, a minimum 6-foot sidewalk width should be used, exclusive of the width of the curb.

   Curb inlets should also be taken into consideration when determining buffer width. The minimum buffer width should be provided in order to provide a sidewalk surface that is non-vibratory and has no vertical discontinuities due to settlement.

   Where signs are proposed within the buffer area, a wider buffer should be designed to provide adequate clearance from the edge of signs. A minimum two feet of clearance should be provided between the edge of the sign and the edge of the sidewalk as shown in Detail “A” of Figure 14-2-C. A minimum two feet of clearance should also be provided between the edge of any sign and the face of curb.

A minimum 1-foot buffer area should also be provided between the sidewalk and adjacent side slopes; however, wider buffer areas are recommended to provide greater separation from adjacent drop-offs where safety rail would otherwise be warranted. See Figure 14-2-E.

5. Buffer Area Without Curb – Sidewalks adjacent to roadways should be separated from the roadway by a curb. However, in some situations, improvements to pedestrian facilities may be proposed where the installation of curb and gutter is not feasible. On-street parking can serve as an acceptable buffer without the presence of curb and gutter, but where no on-street parking exists, the minimum buffer width between the edge of pavement and the edge of sidewalk should be five feet. Where sufficient distance is not available to provide this width, a physical barrier or railing should be provided between the roadway and the sidewalk to prevent sidewalk users from making undesirable or unintended movements toward the roadway, and to reinforce the concept that the sidewalk is an independent facility. The physical barrier or railing does not need to be crashworthy, should not impair sight distance at intersections, and should be designed to limit the potential for injury to out-of-control drivers

6. Curb Ramps – The *Standard Drawing*s provide criteria for the design of curb ramps, which allow pedestrians to safely transition between the sidewalk, roadway, and median islands. Curb ramps shall be installed at intersections with a ramp at each end of each crosswalk, mid-block crossings (including trail crossings), disabled on-street parking spaces, passenger loading zones, and bus stops. Curb ramps should preferably be aligned with crosswalks such that wheelchair users can more easily orient themselves to cross the street. Where adjacent pedestrian facilities do not exist but are anticipated in the future, curb ramps should be installed to serve occasional pedestrians until such time as a sidewalk is constructed.

7. Driveways – The *Standard Drawings* provide details for the treatment of sidewalks at driveways and intersections. Where the pedestrian access route is proposed adjacent to the curb, special designs may be necessary at driveways to maintain the recommended sidewalk criteria for width, grade, and cross slope. Section 14-2.07.02 discusses the design of driveway entrances.

8. Sidewalk Design on Bridges – The sidewalk width on a bridge shall be the same as the sidewalk width on the roadway approach; however, the buffer width (see Item 11 below) may be omitted across the bridge. Where the elevation of the proposed sidewalk on the bridge and roadway differ, the sidewalk shall be transitioned outside the ends of the bridge at a rate no steeper than 20:1.

   Along bridges on low-speed roadways (V ≤ 45 miles per hour), a vertical curb at the edge of the sidewalk is usually sufficient to separate pedestrians from vehicular traffic. For high-speed roadways, a barrier-type rail should be provided on bridges to separate the sidewalk from the traveled way. See Section 9-7.02 for more details.

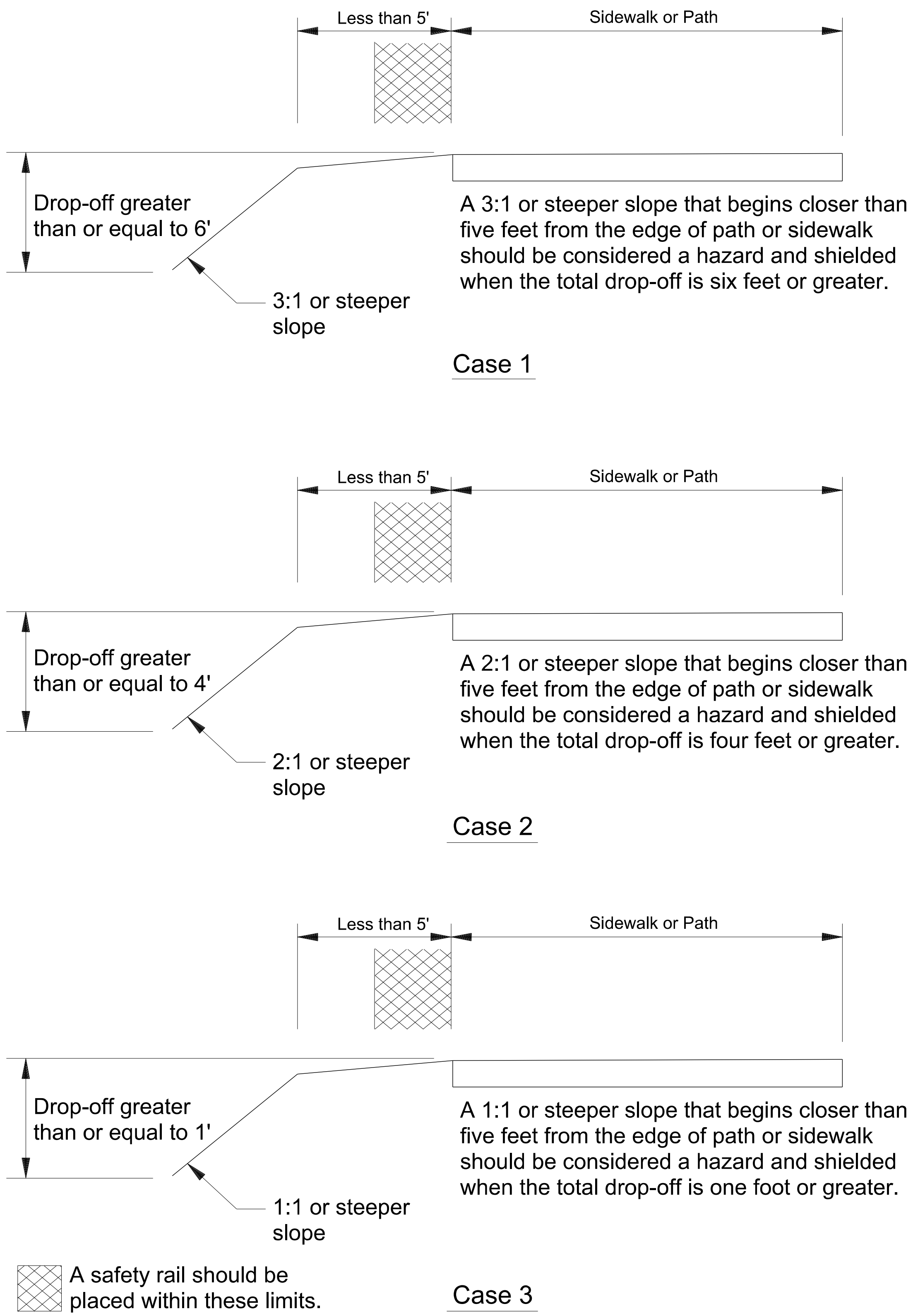
9. Grade – Sidewalks shall not have a profile that exceeds 5% unless the sidewalk is adjacent to an existing roadway. In such cases, sidewalks are allowed to match the roadway profile, even where it exceeds 5%.

10. Cross Slope – Sidewalks shall not have a cross slope that exceeds 2.0%. The maximum design sidewalk cross slope should be 1.5% to provide tolerances for construction activities. The minimum sidewalk cross slope should be 1.0% to facilitate adequate drainage.

11. Surface – The sidewalk surface treatment can have a significant impact on the overall accessibility and comfort level of the facility. Sidewalk surfaces shall be continuous without vertical discontinuities, non-vibratory, stable, firm, and slip resistant. Where vertical discontinuities are unavoidable, any such discontinuities shall be in accordance with the maximum amount as described in the *PROWAG*.

    The sidewalk should be constructed with a material that requires minimal maintenance. The preferred material is Portland cement concrete. Decorative surfaces composed of small units such as stamped concrete or bricks may be undesirable for people using wheelchairs, strollers, carts, or wheeled baggage. The use of these materials should be limited to areas outside of the intended pedestrian access route. However, textured materials used as accents at the edge of the walkway can serve as a tactile edge line to assist pedestrians with visual disabilities while still providing decorative character.

12. Drop-off – Safety rail, sometimes referred to as pedestrian guardrail, may be recommended along sidewalks at certain locations to protect pedestrians from vertical drop-offs or steep slopes. Figure 14-2-E illustrates details for determining locations where safety rail is recommended. Safety rail should also be considered to protect pedestrians from the presence of traffic or a body of water at the bottom of an adjacent slope, or adjacent sloped surfaces consisting of rough materials such as riprap.



TREATMENT FOR SIDEWALK DROP-OFFS
Figure 14-2-E

14-2.06.06 On-Street Parking

Existing and developing adjacent land uses along urban roadways may justify the consideration of on-street parking to provide convenient access for drivers to businesses and residences. In many communities, codes and ordinances dictate the dimensions for parking layout. This subsection contains information for on-street parking where code/ordinance direction may not be available. The *ITE Traffic Engineering Handbook* should be referenced for additional information.

Desirably, parking needs should be met with off-street parking. If adequate off-street parking facilities are not available, on-street parking may then be considered. However, on-street parking may decrease through-traffic capacity and impede traffic flow. Therefore, traffic capacity and local access needs should be balanced when deciding where and when to permit on-street parking.

The two types of on-street parking are parallel parking and angled parking. Angled parking provides more spaces per linear foot than parallel parking, but a greater cross-street width is necessary for its design. Additionally, angled parking requires the vehicle to back into the travel lane where the sight distance may be restricted by adjacent parked vehicles. Therefore, angled parking should preferably be used only on local roadways and on low-speed and low-volume, commercially-oriented arterials and collectors. See Figure 14-2-F for parallel parking configurations.

Parallel parking is preferred where street space is usually limited and traffic capacity is a key factor. However, parallel parking requires a vehicle to stop in the travel lane and await an opportunity to back into the parking space. The total entrance and exit time for parallel parking exceeds that required for angled parking. See Figure 14-2-G for angled parking configurations.

The following information should be applied when on-street parking is being considered:

1. Parking shall be such that the vehicles cannot protrude into the pedestrian access route.
2. On-street parking should be prohibited on major roadways with speeds greater than 35 miles per hour due to potential conflicts associated with door openings and maneuvering in and out of spaces.
3. Removal of existing parking should be coordinated with local officials and affected commercial establishments.
4. Bicycle lanes should not be placed adjacent to conventional front-in angled parking because drivers have poor visibility of bicyclists in the bicycle lane.
5. Bicycle lanes should be placed between the traveled way and the parallel parking lane, and not between the parking lane and curb.
6. The *PROWAG* should be referenced for additional information on accessibility requirements for parking.
7. The location of parking stalls should be prohibited as follows:
    a. within 20 feet of any crosswalk

b. within 20 feet of the beginning of the curb radius at intersections
c. within 10 feet of the beginning of the curb radius at mid-block driveway entrances
d. within 30 feet of any signalized intersection
e. within 50 feet of the nearest rail of a railroad/highway crossing
f. at areas designated by local traffic and enforcement regulations (e.g., near fire hydrants, loading zones, bus stops). Local ordinances should be reviewed for additional information on parking restrictions.

8. Figures 14-2-F and 14-2-G provide the width and length criteria for parallel parking and angled parking stalls, respectively. Figure 14-2-G also indicates the number of stalls that can be provided for each angle for a given curb length. The following additional information should be applied when determining the dimensions of parking stalls:

    a. The geometric design criteria tables in Section 14-2.08 provide the parking lane widths for parallel parking. For angled parking, the parking lane widths should desirably be a combination of “A” and “B” as shown in Figure 14-2-G, exclusive of the through travel lane. However, in restricted areas, a portion of the “B” dimension may be proposed for the through travel lane, thereby reducing the actual parking lane widths.

    b. The effects of parking stalls on through traffic in adjacent travel lanes should also be considered. Drivers will shy away from parked vehicles as they move along an urban roadway. For parallel parking, this characteristic is reflected in the parking lane widths provided in the geometric design criteria tables. For angled parking, the travel lane widths in the geometric design criteria tables should be increased by two feet to account for the shy distance.

    c. The width of gutter pans, if provided, may be considered part of the parking lane width.

    d. As indicated in Figure 14-2-G, parked vehicles should be provided a certain distance “B” to back out of or to pull into their stalls.

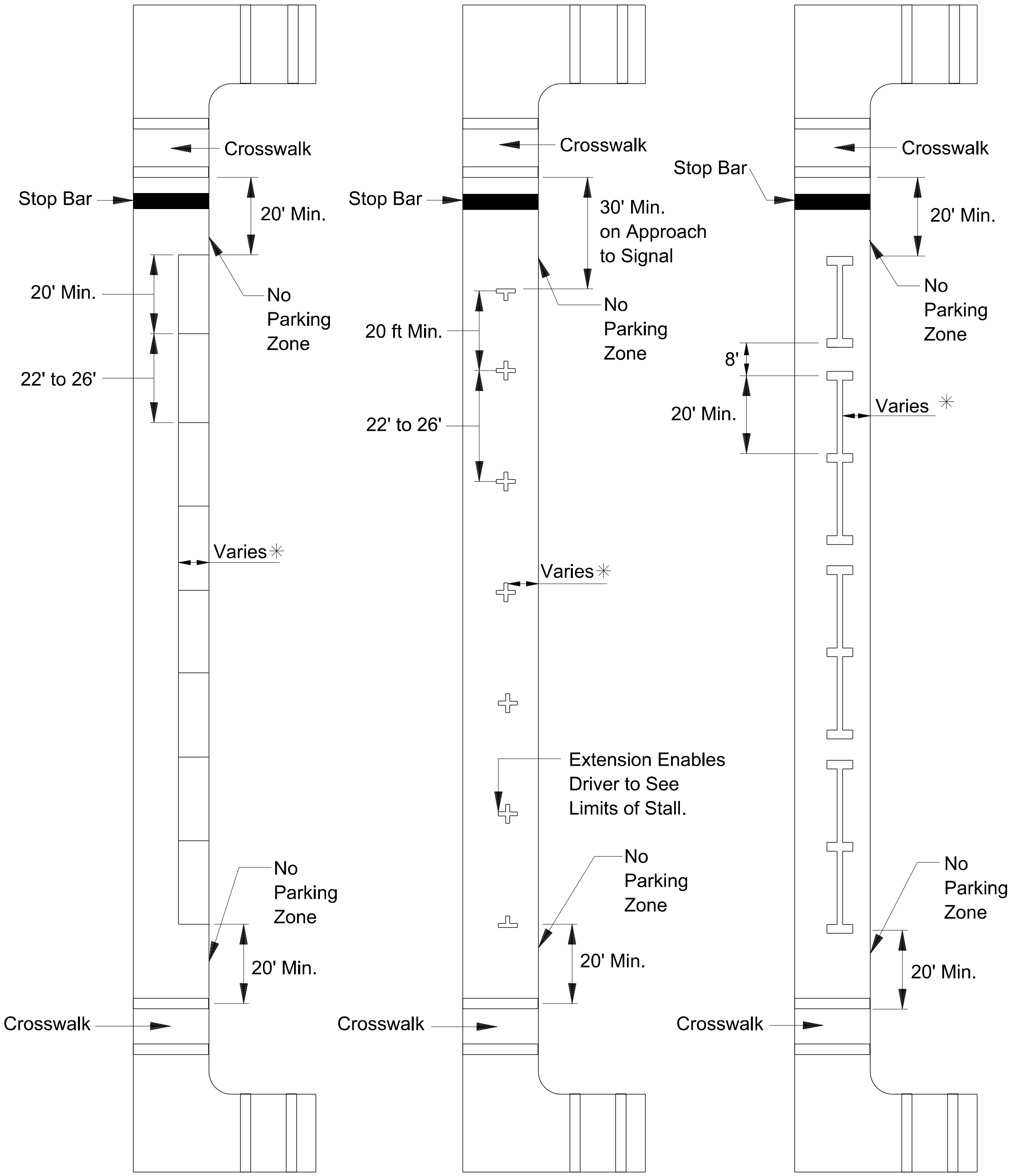
The following additional guidelines should be considered:

1. On-street parking should be located based on the characteristics of the urban street, needs of the adjacent land uses, applicable local policies, and plans for parking management.

2. Curb parking on urban arterial roadways may be acceptable when the available through-traffic lanes can reasonably accommodate traffic demand.

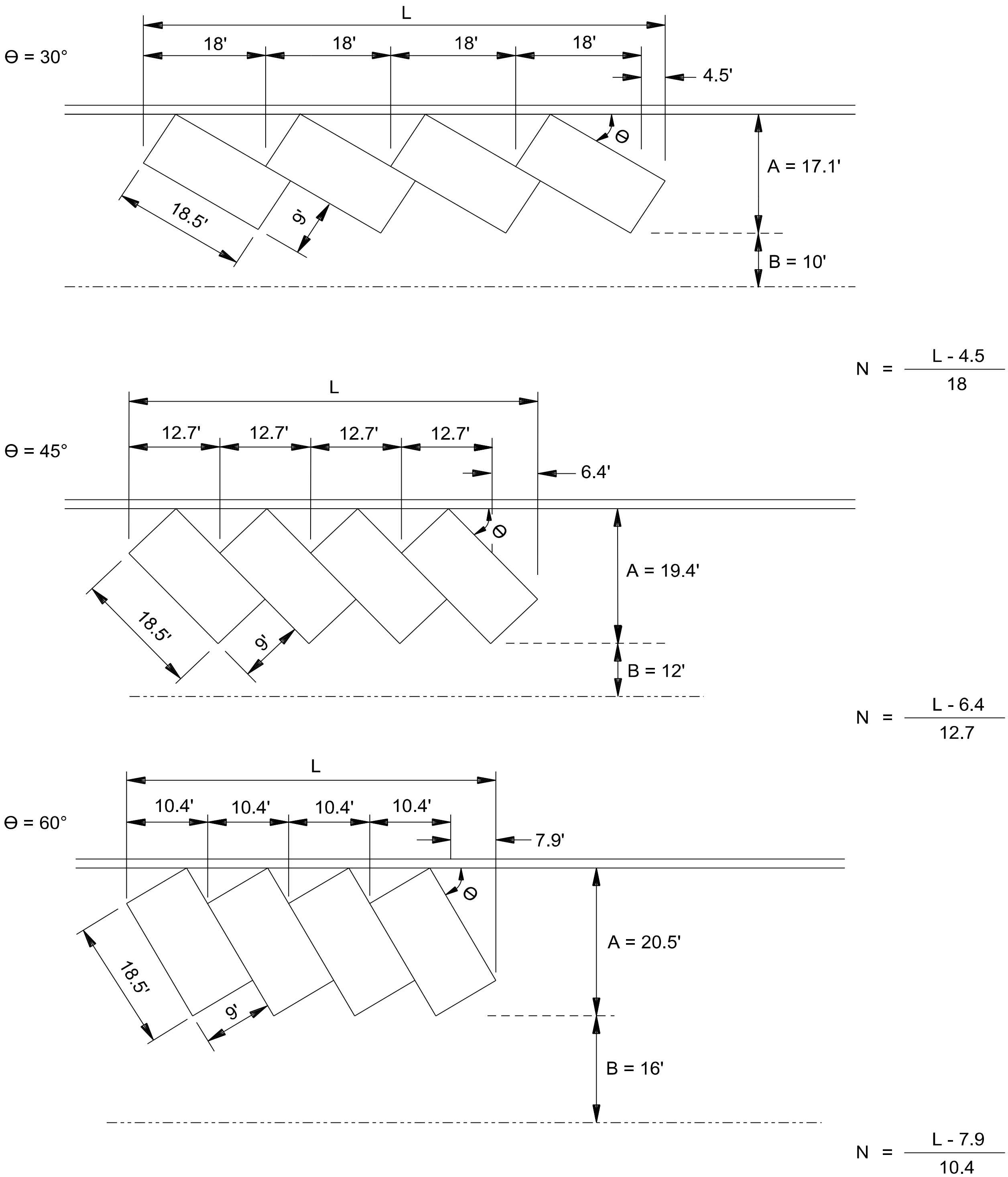
3. The width of the parking lane is dependent upon traffic conditions, functional classification, and the anticipated frequency of parking turnover.

4. The cross slope of the parking lane should typically be the same as the adjacent travel lane.



✳ See Section 14-2.08 for parking lane widths.

PARALLEL CURB PARKING CONFIGURATIONS
(On-Street Parking)
Figure 14-2-F



See next page for notes to figure.

ANGLED CURB PARKING CONFIGURATIONS
(On-Street Parking)
Figure 14-2-G

Notes:

L = Given curb length with parking spaces, feet

N = Number of parking spaces over distance "L"

A = Distance between face of curb and back of stall, assuming that bumper of parked car does not extend beyond curb face, feet

In restricted locations, it can be assumed that the car will move forward until its tire contacts the curb. In these cases, the "A" distances in the figure may be reduced as shown in the table below. However, this reduction is allowable only if an adequate buffer width is provided such that vehicles do not protrude into the pedestrian access route.

Angle of Parking	*Reduction in "A"*
30°	*1.2 feet*
45°	*1.8 feet*
60°	*2.2 feet*

B = Minimum clear distance needed for a parked vehicle to back out of stall while clearing adjacent parked vehicles, feet

14-2.06.07 Medians

The principal functions of a median are to provide a separation from opposing traffic, to prevent undesirable turning movements, to provide pedestrian refuge, and to provide width for future lanes. Medians on urban roadways may be raised, flush, or depressed. Section 6-8.0, the *Access Management Manual*, and AASHTO's *A Policy on Geometric Design of Highways and Streets* provides additional information on medians.

14-2.06.08 Bridges

The geometric design criteria tables in Section 14-2.08 provide the criteria for the minimum width of new and reconstructed bridges on urban roadways. The structural design of bridges should be according to the AASHTO's *LRFD Bridge Design Specifications*, latest edition. The minimum design loading structural capacity for new and reconstructed bridges is HL-93.

For a reconstruction project, there may be one or more existing bridges within the project limits. The geometric design criteria tables in Section 14-2.08 provide the minimum bridge width and design loading structural capacity for existing bridges to remain in place. If a bridge is structurally sound and it meets the design loading structural capacity, it is unlikely to be cost efficient to improve the geometrics of the bridge.

14-2.06.09 Roadside Safety

An adequate lateral offset should be provided between the traveled way and roadside obstructions on urban roadways, which should be free of all unyielding objects, including sign supports, light poles, utility poles, trees, and any other fixed objects. However, limited right of way often restricts the available lateral offset for urban roadways.

Where curbs are not present, the clear zone distance as discussed in Section 9-2.0 should be provided. Clear zone distances are based on design speed, traffic volumes, and side slopes. Where the recommended clear zone distances cannot be met, a 12-foot lateral offset (measured from edge of traveled way) should be provided on the outside of horizontal curves, and an 8-foot lateral offset should be provided elsewhere.

Where curbs are present, a clear zone distance as discussed in Section 9-2.0 should desirably be provided, but this distance is often not feasible for projects on existing roadways. In such cases, objects should ideally be located as far away as feasible. However, where right of way and/or environmental constraints exist, a minimum 1.5-foot lateral offset (measured from the face of curb) should be provided. Breakaway designs should be used for poles and appurtenances located less than six feet from the face of the curb. Section 6-2.0 should be referenced for information on lateral offsets for at-grade intersections.

For additional information, see Chapter 9, “Roadside Safety”.

14-2.06.10 Right of Way Width

The minimum right of way width for urban roadways is the sum of the various cross-section elements, including traveled way width, shoulder widths, median width, side slopes, clear zones, public utilities, border widths, and frontage roads where provided. A wider right of way width may be desirable to allow for future expansion. Right of way widths in developed urban areas are governed primarily by economic considerations, physical obstructions, city ordinances pertaining to minimum set-back requirements, or environmental constraints.

14-2.07 <u>At-Grade Intersections</u>

14-2.07.01 Application of Chapter 6, “At-Grade Intersections”

Chapter 6 presents the criteria for the design of at-grade intersections for urban and rural roadways. The criteria in Section 6-2.0 apply directly to urban roadway projects; however, it may not always be feasible to meet these criteria on existing urban roadways due to right of way and/or environmental constraints. In such cases, intersections that warrant improvements should be improved as much as feasible.

14-2.07.02 Urban Driveways

Urban driveways should be restricted to locations where vehicular movements into and out of driveways will minimize disruption to the flow of traffic on the major roadway. Driveway location and design are related to the functional classification of the roadway and the volume and type of traffic using the facility.

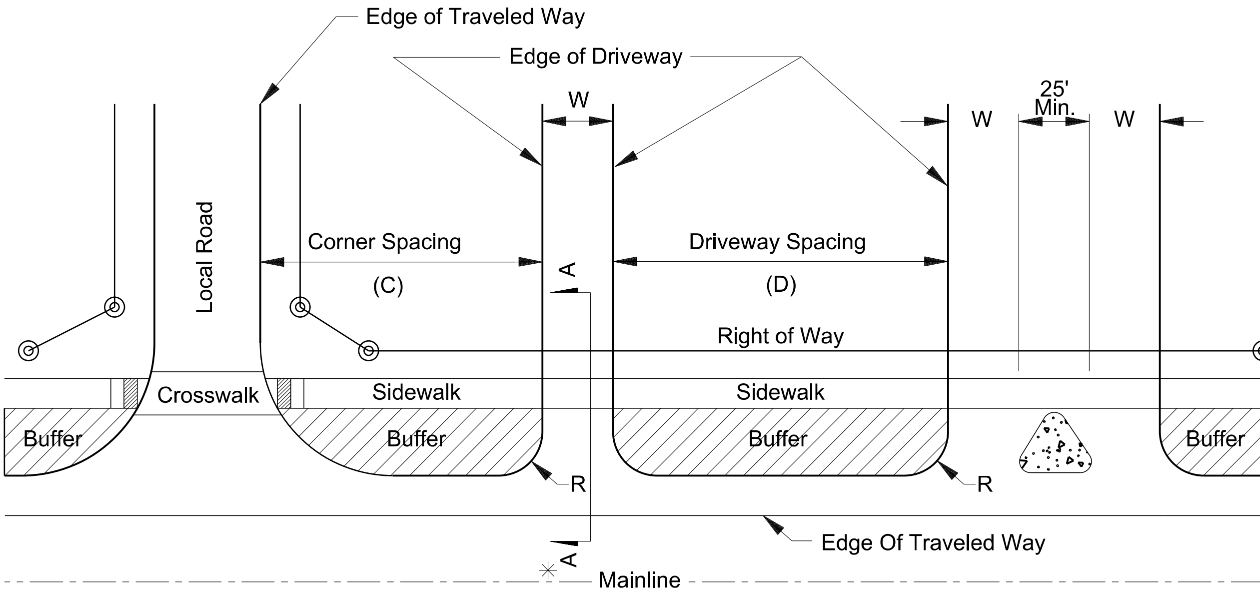
Table 14-2-D and Figures 14-2-H, 14-2-I, and 14-2-J present design criteria for urban driveways. The *Standard Drawings* and *Access Management Manual* should be referenced for additional design details.

Table 14-2-D
RECOMMENDED DESIGN CRITERIA FOR URBAN DRIVEWAYS (2-WAY TRAFFIC)

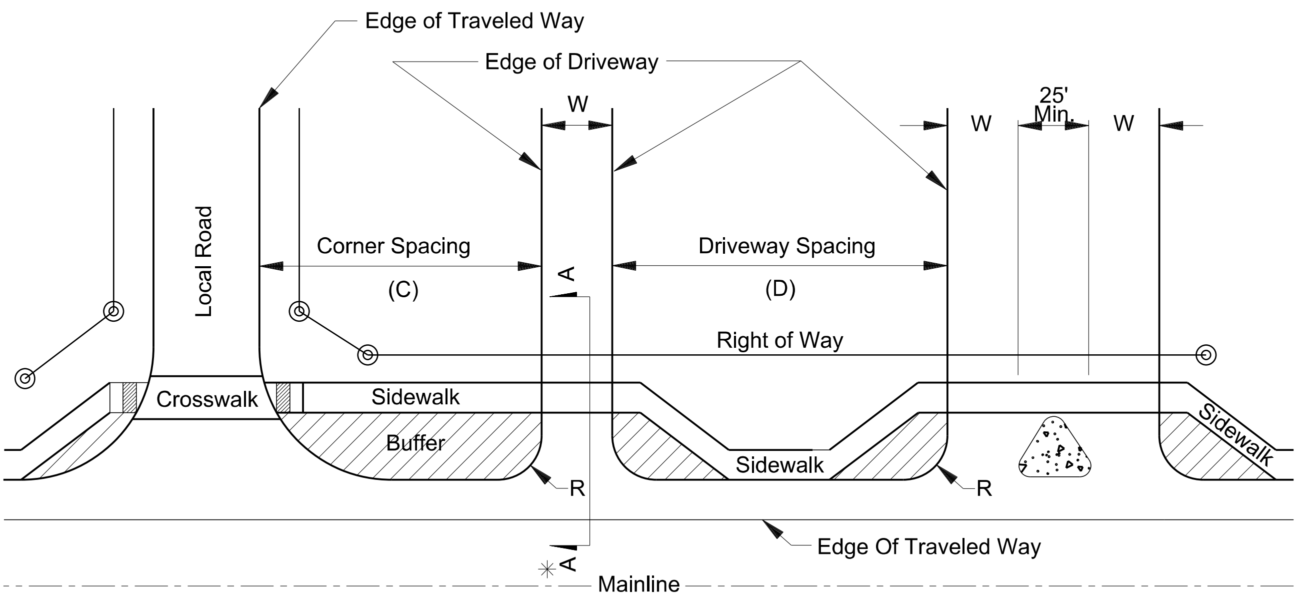
Driveway Design Element		Driveway Type		Functional Classification		
				Arterial	Collector	Local Road
Corner Spacing (C)		All		**(1)**		
Driveway Spacing (D)		All		**(2)**		
Turning Radii (R)		Residential		5 ft to 10 ft **(3)**		
		Commercial/ Industrial		30 ft to 50 ft **(4)**		
Driveway Width (W) **(5)**		Residential		Minimum: 16 ft		
		Commercial/ Industrial		Minimum: 30 ft Maximum: 50 ft		
Maximum Grades	G_1	All	Without Curbs	Cut Section: 2% – 4% (down) Fill Section: Shoulder Slope (4% typical) (down)		
			With Curbs	10% or Flatter		
	G_2	All		15% **(6)**		
Change in Grade without Vertical Curve (ΔG)	At Driveway Entrance with Curbs	All		Maximum: 21%		
	All Other Locations			Maximum: 12%		
Driveway Side Slopes (without curbs)		All		Maximum: 3:1		
Sight Distance		All		**(7)**		

Note: Illustrations of terms are shown in Figure 14-2-H and Figure 14-2-J.

1. *Corner Spacing (C) – The Access Management Manual provides the minimum corner clearance requirements. No part of a driveway entrance or exit should be permitted within the turning radius of an intersection (Section 6-2.0) or within the sight triangles (flares) of an intersection (Section 6 7.04). If feasible, greater distances for corner spacing should be provided.*
2. *Driveway Spacing (D) – The Access Management Manual provides the minimum connection spacing for driveways, which varies based upon traffic, posted speed, and whether the driveway is commercial or residential.*
3. *Turning Radii (R) (Residential) – The preferred minimum turning radius (measured from face of curb) is five feet. However, a 3-foot radius may be used with wider driveways or for driveways on local roadways. All proposed designs should be checked to ensure that the design vehicle has adequate room to make the turns.*
4. *Turning Radii (R) (Commercial/Industrial) – Recommended turning radii are based on a WB-67 design vehicle. Turning radii outside of the recommended range may be warranted based on the selected design vehicle, driveway width, or surrounding constraints. All proposed designs should be checked to ensure that the design vehicle has adequate room to make the turns.*
5. *Driveway Widths (W) – All proposed designs should be checked to ensure that the design vehicle has adequate room to make the turns. For 2-way operation, the minimum width for multi-unit residential driveways should be 30 feet. For commercial/industrial driveways with 2-way operation, a driveway width should be designed based on vehicles entering and exiting from the driveway simultaneously.*
6. *Maximum Grades (G2) – In restricted locations, G2 may be as steep as 20%.*
7. *Intersection Sight Distance (ISD) – Major commercial driveways or multi-unit residential driveways should preferably be designed to meet the ISD criteria set forth in Section 6-6.0. Additionally, the recommended SSD should be provided for all driveways. Sight obstructions (e.g., large trees, landscaping, signs, mailboxes) within the sight triangle at the driveway entrance should be cleared or relocated. To perform the check, it is reasonable to assume an eye location of approximately 10 feet from the edge of traveled way.*



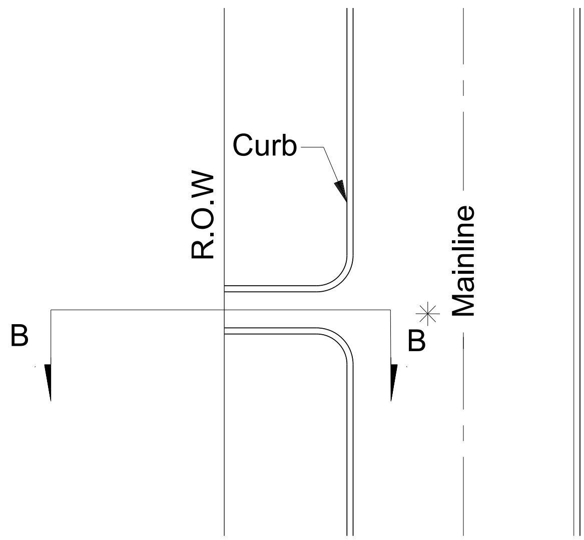
(a) Plan View with Constant Buffer



(b) Plan View without Buffer or with Variable Buffer

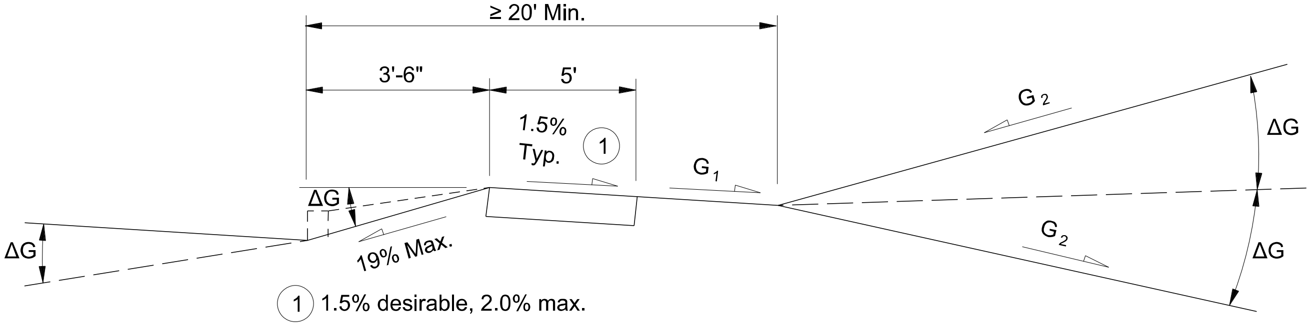
✳See Figure 14-2-J for Section A-A.

TYPICAL URBAN DRIVEWAYS
Figure 14-2-H



✳ See Figure 14-2-J for Section B-B.

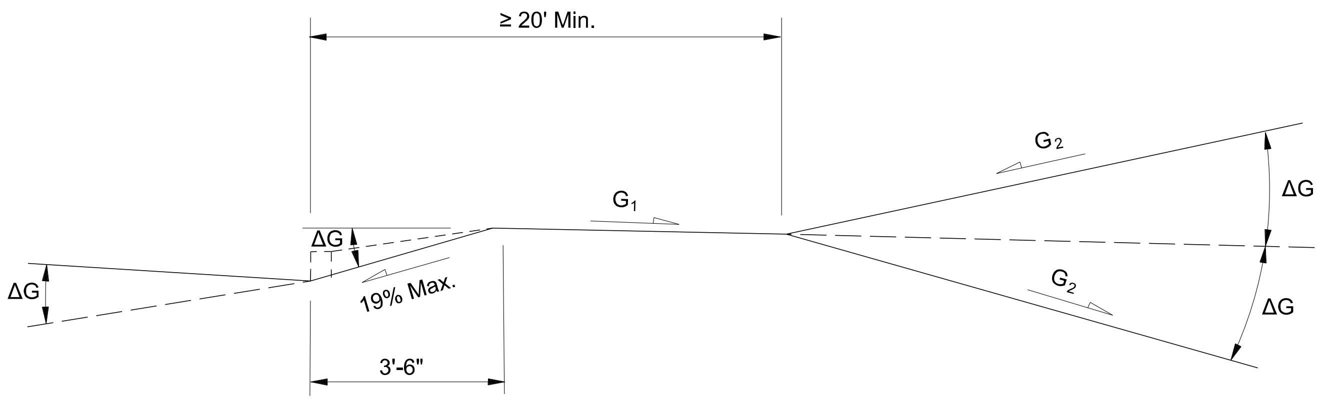
PLAN VIEW OF URBAN DRIVEWAY
(Without Sidewalk)
Figure 14-2-I



Section A-A ✳

(a) Profile With Curb and Sidewalk

✳ See Figure 14-2-H for plan view.



Section B-B ✳✳

(b) Profile With Curb And No Sidewalk

✳✳ See Figure 14-2-I for plan view.

TYPICAL URBAN DRIVEWAY PROFILES
Figure 14-2-J

14-2.08 Geometric Design Criteria

This section presents the Department's criteria for the design of urban roadways on new and reconstruction projects. Detailed information for alignment and cross-section elements can be found in Section 2-8.02 and 2-8.03, respectively, while design criteria for each can be found in the following locations:

- Table 14-2-E (freeways)
- Tables 14-2-F and 14-2-G (arterials)
- Tables 14-2-H and 14-2-I (collectors)
- Table 14-2-J (local roadways)

Each table is followed by a set of footnotes that provides additional information for the items listed in the geometric design criteria tables.

The criteria provided in Table 14-2-C, as well as the additional information provided in the footnotes for this table, meet or exceed the criteria found in the AASHTO publication *A Policy on Design Standards – Interstate System*. The design for all urban interstates shall be in compliance with these criteria.

When the design criteria proposed for new construction or reconstruction projects do not comply with the values presented in the above-referenced tables, see Design Exception and Design Variance procedures presented in Section 2-10.0.

Table 14-2-E
GEOMETRIC DESIGN CRITERIA FOR URBAN FREEWAYS
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	Urban
Design Controls	Design Year			2-3.02	20 Years
	*Design Speed			2-2.01	60 mph – 70 mph
	Control of Access			11-1.05	Full (Type 1)
	Level of Service Threshold			2-3.04	D
Cross-Section Elements	*Travel Lane Width			2-8.03	12 ft
	Outside Shoulder Width **(1)**		*Usable	2-8.03	12 ft
			*Paved		10 ft
	Median Shoulder Width **(1)**		*Usable	2-8.03	8 ft
			*Paved		4 ft
	*Cross Slope		Travel Lane **(2)**	2-8.03	2%
			Shoulder		See Note **(3)**
	Auxiliary Lanes		Lane Width	2-8.03	12 ft
			Shoulder Width		Paved: 10 ft Usable: 12 ft
	Median Width		Depressed	2-8.03	64 ft
			Concrete Median Barrier		≥ 4 Lanes: 26 ft Desirable; 22 ft Minimum
	New and Reconstructed Bridges		*Design Loading Structural Capacity	2-8.03	HL-93
			Minimum Width **(4)**		Traveled Way + 12 ft (outside shoulder) + 6 ft (median shoulder)
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	2-8.03 12-4.02	See Note **(5a)**
			Minimum Width **(5b)**		Traveled Way + 10 ft (outside shoulder) + 4 ft (median shoulder)
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			11-1.01 14-2.06	30 ft
	Roadside Clear Zone		Guardrail	9-2.0 14-2.06	Usable Shoulder Width
			Obstruction **(6)**		30 ft
	Slope Schedule **(7)**	Cut	Foreslope (within clear zone)	2-8.03	6:1
			Depth of Ditch		4 ft
			Backslope		3:1
		Fill	Safety Slope (within clear zone)		6:1
			Fill Slope (outside clear zone)		3:1

	DESIGN SPEED		Manual Section	60 mph	65 mph	70 mph
Alignment Elements	*Stopping Sight Distance		2-8.02	570 ft	645 ft	730 ft
	Decision Sight Distance **(8)**		2-8.02 4-5.02	1280 ft	1365 ft	1445 ft
	*Superelevation Rate		3-4.01	e_{max} = 10%		
	*Minimum Horizontal Curve Radius		3-3.0	1090 ft	1340 ft	1630 ft
	*Maximum Grades **(9)**	Level	4-3.01	3%	3%	3%
		Rolling		4%	4%	4%
	Minimum Grades		4-3.02 14-2.05	See Note **(10)**		
	Vertical Curve (K-values)	*Crest	4-5.0	151	193	247
		Sag		136	157	181
	*Vertical Clearance (freeway under)	New/Reconstructed Bridges **(11)**	2-8.02 2-10.03	Desirable: 17 ft Minimum: 16 ft		
		Existing Bridges		Desirable: 17 ft Minimum: 16 ft		
		Sign Truss/ Pedestrian Bridge		19 ft		
	Vertical Clearance (freeway over railroad) **(12)**		2-8.02	25 ft		

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Table 14-2-E

8. *<u>Shoulder Width</u> –*

   a. *The minimum paved shoulder widths shown in Table 14-2-E are consistent with the AASHTO publication* A Policy on Design Standards – Interstate System. *Paved shoulder widths on interstates shall not be less than these widths. The table below includes these widths, but also provides additional information regarding usable shoulder widths, freeways with three or more lanes in one direction, and truck traffic.*

No. of Lanes in One Direction	*Left*		*Right*	
	Usable	*Paved*	*Usable*	*Paved*
2	*8 ft*	*4 ft*	*12 ft**	*10 ft**
3 or more	*12 ft**	*10 ft**	*12 ft**	*10 ft**

   **Where truck traffic exceeds 250 DDHV, consideration should be given to increasing these widths by two feet.*

   b. *Paved shoulders on non-interstate freeways wider than the typical 2-foot width should not be provided except as approved for special conditions.*

   c. *Where the two roadways are separated by a CMB, the desirable paved median shoulder width is 12 feet; the minimum paved median shoulder width is 10 feet. Where the truck traffic exceeds 250 DDHV, the 12-foot width should be used.*

   d. *A minimum paved shoulder width of 10 feet, desirably 12 feet, should be provided for a distance of 350 feet beyond a reduction of the basic number of lanes (lane drop) for a recovery area for drivers. See Section 7-2.05 for more information.*

9. *<u>Cross Slope of Travel Lane (normal crown sections)</u> – The typical cross slope of the traveled way in a normal crown section is 2.0% and shall not be less than 1.5%. For freeways with three or more lanes sloped in the same direction, the cross slope may be increased to 2.5% for all lanes beyond the first two lanes that are sloped in the same direction. The cross slope of an outer lane shall not be less than that of the adjacent lane.*

10. *<u>Cross Slope of Shoulder (normal crown sections)</u> – The cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%. The portion of the shoulder that is not paved should have a cross slope of 4%. The cross slope of the shoulder shall not be less than that of the adjacent lane.*

11. *<u>New and Reconstructed Bridge Width</u> –*

    a. *The bridge width shown in the table applies to a 1-way bridge of a divided freeway. The bridge width should be increased for auxiliary lanes and for wider shoulders as shown in Footnote 1 above.*

   b. *For bridges less than or equal to 200 feet in length, the bridge width shall be no less than the full paved width of the approach roadway when the minimum width provided in the table cannot be met.*

12. *Existing Bridges to Remain in Place –*

   a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

   b. *The width provided in the table applies to a 1-way existing bridge that is less than or equal to 200 feet in length. For existing 1-way bridges that are greater than 200 feet in length, the minimum shoulder width shall be no less than four feet for both the left and the right sides.*

   c. *For existing bridges that are in compliance with the applicable widths as outlined in the table or in footnote 5b, consideration should be given to widening the bridges to comply with the new construction width, especially in areas with high traffic volumes.*

13. *Roadside Clear Zone – The width shown in the table is for design speeds of 60 mph to 70 mph, for 6:1 fill slopes or cut foreslopes, and for ADTs greater than 6000. Section 9-2.0 provides clear zone distances for other roadside conditions. All values are measured from the edge of the traveled way, except where long (greater than 0.5 miles) auxiliary lanes are present.*

14. *Slopes – If high-volume-change (HVC) soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

15. *Decision Sight Distance (DSD) – The distance shown in the table is for a speed/path/direction change on an urban freeway.*

16. *Maximum Grades – Grades 1% steeper may also be used in restricted urban areas where development precludes the use of flatter grades.*

17. *Minimum Grades –*

   a. *On roadways, level grades are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

   b. *On bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

18. *Vertical Clearance (freeway under) –*

   a. *The vertical clearance shown in the table shall be provided over the entire freeway width, including shoulders, interchange ramps, and collector-distributor (C-D) roadways.*

   b. *The desirable clearance allows for future resurfacing with additional structure depth.*

   c. *For crossing routes going under the freeway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

d. Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).

19. <u>Vertical Clearance (freeway over railroad)</u> – The vertical clearance shown in the table should typically be provided over the entire railroad right of way width.

Table 14-2-F
GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIALS (2-LANE/3-LANE)
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	With Curb	Without Curb
Design Controls	Design Year			14-2.01	20 Years	
Design Controls	*Design Speed **(1)**			14-2.02	40 mph – 50 mph	45 mph – 55 mph
Design Controls	Control of Access			11-1.05	Control by Regulation (Type 3)	
Design Controls	Level of Service Threshold			14-2.01	Desirable: C Minimum: D	
Cross-Section Elements	*Travel Lane Width **(2)**			14-2.06	12 ft	
Cross-Section Elements	Shoulder Width	*Usable **(3)**		14-2.06	Des: 8 ft Min: 6 ft	8 ft
Cross-Section Elements	Shoulder Width	Paved		14-2.06	Same as Usable	See Note **(4)**
Cross-Section Elements	*Cross Slope	Travel Lane **(5)**		14-2.06	2%	
Cross-Section Elements	*Cross Slope	Shoulder		14-2.06	Shoulder Width > 4 ft: 4% Shoulder Width ≤ 4 ft: 2%	See Note **(6)**
Cross-Section Elements	Auxiliary Lanes	Lane Width		14-2.07	Same as Travel Lane	
Cross-Section Elements	Auxiliary Lanes	Usable Shoulder Width		14-2.07	Des: 4 ft Min: 2 ft	Same as Mainline Shoulder
Cross-Section Elements	CTWLTL Width **(7)**			6-4.03	Desirable: 14 ft Minimum: 12 ft	
Cross-Section Elements	Parking Lane Width **(8)**			14-2.06	Desirable: 12 ft Minimum: 10 ft	
Cross-Section Elements	Sidewalk Width			14-2.06	Minimum: 5 ft	
Cross-Section Elements	New and Reconstructed Bridges	*Design Loading Structural Capacity		14-2.06	HL-93	
Cross-Section Elements	New and Reconstructed Bridges	Minimum Width **(10)**		14-2.06	Approach Roadway Width	
Cross-Section Elements	Existing Bridges to Remain in Place	*Design Loading Structural Capacity		12-3.0 14-2.06	See Note **(11)**	
Cross-Section Elements	Existing Bridges to Remain in Place	Minimum Width		12-3.0 14-2.06	See Note **(11)**	
Cross-Section Elements	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			11-1.01 14-2.06	See Note **(12)**	
Cross-Section Elements	Roadside Clear Zone	Guardrail		9-2.0 14-2.06	Usable Shoulder Width	
Cross-Section Elements	Roadside Clear Zone	Lateral Offset		9-2.0 14-2.06	1.5 ft **(13a)**	See Note **(13b)**
Cross-Section Elements	Slope Schedule **(14)**	Cut	Foreslope	14-2.06	+2%	4:1
Cross-Section Elements	Slope Schedule **(14)**	Cut	Depth of Ditch	14-2.06	1 ft	3 ft
Cross-Section Elements	Slope Schedule **(14)**	Cut	Backslope	14-2.06	Des: 3:1 Max: 2:1	3:1
Cross-Section Elements	Slope Schedule **(14)**	Fill	Height ≤ 10 ft	14-2.06	Des: 3:1 Max: 2:1	4:1
Cross-Section Elements	Slope Schedule **(14)**	Fill	Height > 10 ft	14-2.06	Des: 3:1 Max: 2:1	4:1 (within clear zone) 3:1 (outside clear zone)

	DESIGN SPEED		Manual Section	40 mph	45 mph	50 mph	55 mph
Alignment Elements	*Stopping Sight Distance		14-2.03	305 ft	360 ft	425 ft	495 ft
Alignment Elements	Intersection Sight Distance **(15)**		6-6.0	445 ft	500 ft	555 ft	610 ft
Alignment Elements	*Superelevation Rate		14-2.04	4%		6%	10%
Alignment Elements	*Minimum Horizontal Curve Radius	e_{max} = 4%	14-2.04	533 ft	711 ft	N/A	
Alignment Elements	*Minimum Horizontal Curve Radius	e_{max} = 6%	14-2.04	N/A		833 ft	N/A
Alignment Elements	*Minimum Horizontal Curve Radius	e_{max} = 10%	14-2.04	N/A			877 ft
Alignment Elements	*Maximum Grades	Level	14-2.05	7%	6%	6%	5%
Alignment Elements	*Maximum Grades	Rolling	14-2.05	8%	7%	7%	6%
Alignment Elements	Minimum Grades		4-3.02 14-2.05	See Note **(16)**			
Alignment Elements	Vertical Curve (K-values)	*Crest	14-2.05	44	61	84	114
Alignment Elements	Vertical Curve (K-values)	Sag	14-2.05	64	79	96	115
Alignment Elements	*Vertical Clearance (arterial under)	New/Reconstructed Bridges **(17)**	2-8.02	Desirable: 17 ft Minimum: 16 ft			
Alignment Elements	*Vertical Clearance (arterial under)	Existing Bridges	2-8.02	Desirable: 16 ft Minimum: 14.5 ft			
Alignment Elements	*Vertical Clearance (arterial under)	Sign Truss/Ped. Bridge	2-8.02	19 ft			
Alignment Elements	Vertical Clearance (arterial over railroad) **(18)**		2-8.02	25 ft			

*For application of controlling design criteria, see Section 2-9.02.

Table 14-2-G
GEOMETRIC DESIGN CRITERIA URBAN ARTERIALS (MULTILANE)
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	With Curb	Without Curb
Design Controls	Design Year			14-2.01	20 Years	
	*Design Speed **(1)**			14-2.02	45 mph – 55 mph	45 mph – 60 mph
	Control of Access			11-1.05	Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)	
	Level of Service Threshold			14-2.01	Desirable: C Minimum: D	
Cross-Section Elements	*Travel Lane Width **(2)**			14-2.06	12 ft	
	Outside Shoulder Width		*Usable **(3)**	14-2.06	Des: 10 ft Min: 6 ft	10 ft
			Paved		Same as Usable	8 ft
	Median Shoulder Width		*Usable **(3)**	14-2.06	Des: 4 ft Min: 2 ft	8 ft
			Paved		Same as Usable	3 ft
	*Cross Slope		Travel Lane **(5)**	14-2.06	2%	
			Shoulder		Shoulder Width > 4 ft: 4% Shoulder ≤ 4 ft: 2%	See Note **(6)**
	Auxiliary Lanes		Lane Width	14-2.07	Same as Travel Lane	
			Usable Shoulder Width		Des: 4 ft Min: 2 ft	Same as Mainline Shoulder
	CTWLTL Width **(7)**			6-4.03	Desirable: 14 ft Minimum: 12 ft	
	Parking Lane Width **(8)**			14-2.06	Desirable: 12 ft Minimum: 10 ft	
	Sidewalk Width			14-2.06	Minimum: 5 ft	
	Median Width		Depressed	6-8.02	64 ft	
			Raised	14-2.06	See Note **(9)**	
	New and Reconstructed Bridges		*Design Loading Structural Capacity	14-2.06	HL-93	
			Minimum Width **(10)**		Approach Roadway Width	Traveled Way + 10 ft (outside shoulder) + 6 ft (median shoulder)
	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	12-3.0 14-2.06	See Note **(11)**	
			Minimum Width			
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			11-1.01 14-2.06	See Note **(12)**	
	Roadside Clear Zone		Guardrail	9-2.0	Usable Shoulder Width	
			Lateral Offset	14-2.06	1.5 ft **(13a)**	See Note **(13b)**
	Slope Schedule **(14)**	Cut	Foreslope	14-2.08	+2%	4:1
			Depth of Ditch		1 ft	4 ft
			Backslope		Des: 3:1 Max: 2:1	3:1
		Fill	Height ≤ 10 ft		Des: 3:1 Max: 2:1	4:1
			Height > 10 ft			4:1 (within clear zone) 3:1 (outside clear zone)

	DESIGN SPEED			45 mph	50 mph	55 mph	60 mph
Alignment Elements	*Stopping Sight Distance		14-2.03	360 ft	425 ft	495 ft	570 ft
	Intersection Sight Distance		6-6.0	See Note **(15)**			
	*Superelevation Rate		14-2.04	4%	6%	10%	
	*Minimum Horizontal Curve Radius	e_{max} = 4%	14-2.04	711 ft	N/A		
		e_{max} = 6%		N/A	833 ft	N/A	
		e_{max} = 10%		N/A		877 ft	1090 ft
	*Maximum Grades	Level	14-2.05	6%	6%	5%	5%
		Rolling		7%	7%	6%	6%
	Minimum Grades		4-3.02 14-2.05	See Note **(16)**			
	Vertical Curve (K-values)	*Crest	14-2.05	61	84	114	151
		Sag		79	96	115	136
	*Vertical Clearance (arterial under)	New/Reconstructed Bridges **(17)**	2-8.02	Desirable: 17 ft Minimum: 16 ft			
		Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft			
		Sign Truss/Pedestrian Bridge		19 ft			
	Vertical Clearance (arterial over railroad) **(18)**		2-8.02	25 ft			

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 14-2-F and 14-2-G

1. *<u>Design Speed</u> – A design speed as low as the posted speed limit may be used.*

2. *<u>Travel Lane Width</u> –*

   a. *11-foot lane widths may be used in more constrained areas and where truck and bus volumes are low. The lane widths needed for all lanes and intersection design controls should be evaluated collectively with consideration of all user modes and the adjacent land use.*

   b. *The lane width should be increased by two feet for roadways that include angled on-street parking. See Section 14-2.06.06 for more information.*

3. *<u>Shoulder Width</u> –*

   a. *When using new construction/reconstruction criteria on existing roadways, a minimum 2-foot shoulder width for curbed roadways may be used on the outside shoulder if right of way and/or other constraints exist. For any widths less than the minimum 6-foot width, the drainage design should include consideration of the shoulder width in order to minimize ponding on the roadway.*

   b. *See Section 14-2.06.01 for information on shoulder width relative to curb and gutter usage.*

4. *<u>Shoulder Surface Type</u> – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved in special conditions.*

5. *<u>Cross Slope of Travel Lane (normal crown sections)</u> – The typical cross slope of the traveled way in a normal crown section is 2.0%. For arterials with three or more lanes sloped in the same direction, the cross slope may be increased to 2.5% for all lanes beyond the first two lanes that are sloped in the same direction. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

6. *<u>Cross Slope of Shoulder (normal crown sections)</u> –*

   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

7. *<u>CTWLTL Width</u> – In industrial areas with heavy truck traffic, the CTWLTL width should desirably be 16 feet. Section 6-4.03 provides more information on the CTWLTL width in areas of restricted right of way.*

8. *<u>Parking Lanes</u> – The widths shown in the tables apply only to parallel parking. Section 14-2.06.06 provides information on the design of angled parking.*

9. *<u>Median Width (Raised)</u> – The minimum raised median width on urban roadways should be 18 feet. This width should only be used where the majority of the intersections along the roadway are signalized. If many of the intersections are not signalized, then the recommended width is 22 feet, assuming no right of way restrictions. The 22-foot median width allows passenger vehicles to store within the median crossover. For medians with dual left-turn lanes, the raised median should desirably be 36 feet, but the minimum should be 30 feet.*

10. *<u>New and Reconstructed Bridge Width</u> –*

    a. *The width shown in Table 14-2-G for non-curbed sections applies to a 1-way bridge of a divided arterial. The width should be increased for auxiliary lanes or other features, such as sidewalks or bicycle lanes. For undivided arterials, the width should equal the approach roadway width.*

    b. *For bridges less than or equal to 200 feet in length, the width should be no less than the full paved width of the approach roadway when the minimum widths provided in the tables cannot be met.*

11. *<u>Existing Bridges to Remain in Place</u> –*

    a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

    b. *Undivided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side.*

    c. *Divided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side for each roadway.*

    d. *The minimum widths provided in Footnotes 11b and 11c do not include other existing features, such as auxiliary lanes, sidewalks, and bicycle lanes.*

    e. *For existing bridges that are in compliance with the applicable widths in footnotes 11b, 11c, and 11d, consideration should be given to widening the bridges to comply with the new construction width, especially in areas with high traffic volumes.*

    f. *See Section 14-2.06.05 for information on sidewalks at existing bridges.*

12. *<u>Right of Way Border Width</u> – Right of way border widths should be determined by level of development, property values, etc.*

13. *<u>Roadside Clear Zones</u> –*

    a. *<u>With Curb</u> – Desirably, the clear zone for open shoulder rural sections (Section 9-2.0) should also be provided for curbed sections. The 1.5-foot lateral offset should only be used in constrained areas. The distance is measured from the face of the curb to the obstruction, regardless of shoulder width.*

    b. *<u>Without Curb</u> – The clear zone for open shoulder rural sections (Section 9-2.0) should be provided. Where the recommended clear zone distances cannot be met, a 12-foot lateral offset (measured from edge of traveled way) should be provided on the outside of horizontal curves, and an 8-foot lateral offset should be provided elsewhere.*

14. *<u>Slopes</u> – If high volume change (HVC) soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

15. *<u>Intersection Sight Distance (ISD)</u> – The values provided in Table 14-2-F assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways.*

16. *<u>Minimum Grades</u> –*

    a. *On roadways with curb and gutter, the minimum longitudinal gradient should be 0.2%; however, a grade of 0.4% or greater is desirable to facilitate longitudinal drainage.*

    b. *On roadways without curb and gutter, level gradients are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    c. *On bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

17. *<u>Vertical Clearance (arterial under)</u> –*

    a. *The vertical clearance shown in the table should be provided over the entire arterial roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the arterial roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

18. *<u>Vertical Clearance (arterial over railroad)</u> – The vertical clearance shown in the tables should typically be provided over the entire railroad right of way width.*

Table 14-2-H
GEOMETRIC DESIGN CRITERIA URBAN COLLECTORS (2-LANE/3-LANE)
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	With Curb	Without Curb
Design Controls	Design Year			14-2.01	20 Years	
Design Controls	*Design Speed **(1)**			14-2.02	30 mph – 50 mph	
Design Controls	Control of Access			11-1.05	Control by Regulation (Type 3)	
Design Controls	Level of Service Threshold			14-2.01	Desirable: C Minimum: D	
Cross-Section Elements	*Travel Lane Width **(2)**			14-2.06	12 ft	
Cross-Section Elements	Shoulder Width		*Usable **(3)**	14-2.06	Des: 6 ft Min: 4 ft	6 ft
Cross-Section Elements	Shoulder Width		Paved **(4)**	14-2.06	Same as Usable	See Note **(4)**
Cross-Section Elements	*Cross Slope		Travel Lane **(5)**	14-2.06	2%	
Cross-Section Elements	*Cross Slope		Shoulder	14-2.06	Shoulder Width > 4 ft: 4% Shoulder Width ≤ 4 ft: 2%	See Note **(6)**
Cross-Section Elements	Auxiliary Lanes		Lane Width	14-2.07	Same as Travel Lane	
Cross-Section Elements	Auxiliary Lanes		Usable Shoulder Width	14-2.07	Des: 4 ft Min: 2 ft	Same as Mainline Shoulder
Cross-Section Elements	CTWLTL Width **(7)**			6-4.03	Desirable: 14 ft Minimum: 12 ft	
Cross-Section Elements	Parking Lane Width **(8)**			14-2.06	Desirable: 10 ft Minimum: 8 ft	
Cross-Section Elements	Sidewalk Width			14-2.06	Minimum: 5 ft	
Cross-Section Elements	New and Reconstructed Bridges		*Design Loading Structural Capacity	14-2.06	HL-93	
Cross-Section Elements	New and Reconstructed Bridges		Minimum Width **(10)**	14-2.06	Approach Roadway Width	
Cross-Section Elements	Existing Bridges to Remain in Place		*Design Loading Structural Capacity	12-3.0 14-2.06	See Note **(11)**	
Cross-Section Elements	Existing Bridges to Remain in Place		Minimum Width	12-3.0 14-2.06	See Note **(11)**	
Cross-Section Elements	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			11-1.01 14-2.06	See Note **(12)**	
Cross-Section Elements	Roadside Clear Zone		Guardrail	9-2.0 14-2.06	Usable Shoulder Width	
Cross-Section Elements	Roadside Clear Zone		Lateral Offset	9-2.0 14-2.06	1.5 ft **(13a)**	See Note **(13b)**
Cross-Section Elements	Slope Schedule **(14)**	Cut	Foreslope	14-2.08	+2%	3:1
Cross-Section Elements	Slope Schedule **(14)**	Cut	Depth of Ditch	14-2.08	1 ft	3 ft
Cross-Section Elements	Slope Schedule **(14)**	Cut	Backslope	14-2.08	Des: 3:1 Max: 2:1	3:1
Cross-Section Elements	Slope Schedule **(14)**	Fill	Height ≤ 5 ft	14-2.08	Des: 3:1 Max: 2:1	4:1
Cross-Section Elements	Slope Schedule **(14)**	Fill	Height > 5 ft	14-2.08	Des: 3:1 Max: 2:1	3:1

	DESIGN SPEED		Manual Section	30 mph	35 mph	40 mph	45 mph	50 mph
Alignment Elements	*Stopping Sight Distance		14-2.03	200 ft	250 ft	305 ft	360 ft	425 ft
Alignment Elements	Intersection Sight Distance **(15)**		6-6.0	335 ft	390 ft	445 ft	500 ft	555 ft
Alignment Elements	*Superelevation Rate		14-2.04	4%				6%
Alignment Elements	*Minimum Horizontal Curve Radius	e_{max} = 4%	14-2.04	250 ft	371 ft	533 ft	711 ft	N/A
Alignment Elements	*Minimum Horizontal Curve Radius	e_{max} = 6%	14-2.04	N/A				833 ft
Alignment Elements	*Maximum Grades	Level	14-2.05	9%	9%	9%	8%	7%
Alignment Elements	*Maximum Grades	Rolling	14-2.05	11%	10%	10%	9%	8%
Alignment Elements	Minimum Grades		14-2.05	See Note **(16)**				
Alignment Elements	Vertical Curve (K-values)	*Crest	14-2.05	19	29	44	61	84
Alignment Elements	Vertical Curve (K-values)	Sag	14-2.05	37	49	64	79	96
Alignment Elements	*Vertical Clearance (collector under)	New/Reconstructed Bridges **(17)**	2-8.02	Desirable: 17 ft Minimum: 16 ft				
Alignment Elements	*Vertical Clearance (collector under)	Existing Bridges	2-8.02	Desirable: 16 ft Minimum: 14.5 ft				
Alignment Elements	*Vertical Clearance (collector under)	Sign Truss/Ped. Bridge	2-8.02	19 ft				
Alignment Elements	Vertical Clearance (collector over railroad) **(18)**		2-8.02	25 ft				

*For application of controlling design criteria, see Section 2-9.02.

Table 14-2-I
GEOMETRIC DESIGN CRITERIA URBAN COLLECTORS (MULTILANE)
(New Construction/Reconstruction)

	DESIGN ELEMENT		Manual Section	With Curb	Without Curb
Design Controls	Design Year		14-2.01	20 Years	
	*Design Speed **(1)**		14-2.02	30 mph – 50 mph	
	Control of Access		11-1.05	Control by Regulation (Type 3)	
	Level of Service Threshold		14-2.01	Desirable: C Minimum: D	
Cross-Section Elements	*Travel Lane Width **(2)**		14-2.06	12 ft	
	Outside Shoulder Width	*Usable **(3)**	14-2.06	Des: 8 ft Min: 4 ft	10 ft
		Paved **(4)**		Same as Usable	8 ft
	Median Shoulder Width	*Usable **(3)**	14-2.06	Des: 4 ft Min: 2 ft	8 ft
		Paved **(4)**		Same as Usable	3 ft
	*Cross Slope	Travel Lane **(5)**	14-2.06	2%	
		Shoulder		Shoulder Width > 4 ft: 4% Shoulder Width ≤ 4 ft: 2%	See Note **(6)**
	Auxiliary Lanes	Lane Width	14-2.07	Same as Travel Lane	
		Usable Shoulder Width		Des: 4 ft Min: 2 ft	Same as Mainline Shoulder
	CTWLTL Width **(7)**		6-4.03	Desirable: 14 ft Minimum: 12 ft	
	Parking Lane Width **(8)**		14-2.06	Desirable: 12 ft Minimum: 10 ft	
	Sidewalk Width		14-2.06	Minimum: 5 ft	
	Median Width	Depressed	6-8.02	64 ft	
		Narrow/Raised	14-2.06	See Note **(9)**	
	New and Reconstructed Bridges	*Design Loading Structural Capacity	14-2.06	HL-93	
		Minimum Width **(10)**		Approach Roadway Width	Traveled Way + 10 ft (outside shoulder) + 6 ft (median shoulder)
	Existing Bridges to Remain in Place	*Design Loading Structural Capacity	12-3.0 14-2.06	See Note **(11)**	
		Minimum Width			
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)		11-1.01 14-2.06	See Note **(12)**	
	Roadside Clear Zone	Guardrail	9-2.0	Usable Shoulder Width	
		Lateral Offset	14-2.06	1.5 ft **(13a)**	See Note **(13b)**
	Slope Schedule **(14)**	Cut: Foreslope	14-2.08	+2%	4:1
		Cut: Depth of Ditch		1 ft	4 ft
		Cut: Backslope		Des: 3:1 Max: 2:1	3:1
		Fill: Height ≤ 5 ft		Des: 3:1 Max: 2:1	4:1
		Fill: Height > 5 ft			4:1 (within clear zone) 3:1 (outside clear zone)

	DESIGN SPEED		Manual Section	30 mph	35 mph	40 mph	45 mph	50 mph
Alignment Elements	*Stopping Sight Distance		14-2.03	200 ft	250 ft	305 ft	360 ft	425 ft
	Intersection Sight Distance		6-6.0	See Note **(15)**				
	*Superelevation Rate		14-2.04	4%				6%
	*Minimum Horizontal Curve Radius	e_{max} = 4%	14-2.04	250 ft	371 ft	533 ft	711 ft	N/A
		e_{max} = 6%		N/A				833 ft
	*Maximum Grades	Level	14-2.05	9%	9%	9%	8%	7%
		Rolling		11%	10%	10%	9%	8%
	Minimum Grades		14-2.05	See Note **(16)**				
	Vertical Curve (K-values)	*Crest	14-2.05	19	29	44	61	84
		Sag		37	49	64	79	96
	*Vertical Clearance (collector under)	New/Reconstructed Bridges **(17)**	2-8.02	Desirable: 17 ft Minimum: 16 ft				
		Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft				
		Sign Truss/Ped. Bridge		19 ft				
	Vertical Clearance (collector over railroad) **(18)**		2-8.02	25 ft				

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Tables 14-2-H and 14-2-I

1. *Design Speed – A design speed equal to the posted speed limit may be used.*
2. *Travel Lane Width –*
   a. *11-foot lane widths may be used in more constrained areas and where truck and bus volumes are low. The lane widths needed for all lanes and intersection design controls should be evaluated collectively with consideration of all user modes and the adjacent land use.*

   b. *The lane width should be increased by two feet for roadways that include angled on-street parking. See Section 14-2.06.06 for more information.*
3. *Shoulder Width –*
   a. *When using new construction/reconstruction criteria on existing roadways, a minimum 2-foot shoulder width for curbed roadways may be used on the outside shoulder if right of way and/or other constraints exist. For any widths less than the desirable width, the drainage design should include consideration of the shoulder width in order to minimize ponding on the roadway.*

   b. *See Section 14-2.06.01 for information on shoulder width relative to curb and gutter usage.*
4. *Shoulder Surface Type – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved in special conditions.*
5. *Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0%. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*
6. *Cross Slope of Shoulder (normal crown sections) –*
   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*
7. *CTWLTL Width – In industrial areas with heavy truck traffic, the CTWLTL width should desirably be 16 feet. Section 6-4.03 provides more information on the CTWLTL width in areas of restricted right of way.*
8. *Parking Lanes – The widths shown in the tables apply only to parallel parking. Section 14-2.06.06 provides information on the design of angled parking.*
9. *Median Width (Raised) – The minimum raised median width on urban roadways should be 18 feet. This width should only be used where the majority of the intersections along the roadway are signalized. If many of the intersections are not signalized, then the recommended width is 22 feet, assuming no right of way restrictions. The 22-foot median width allows passenger vehicles to store within the median crossover. For medians with dual left-turn lanes, the raised median width should desirably be 36 feet, but the minimum width should be 30 feet.*

10. <u>New and Reconstructed Bridge Width</u> –

    a. *The width shown in Table 14-2-I applies to a 1-way bridge of a divided collector. The width should be increased for auxiliary lanes or other features, such as sidewalks or bicycle lanes. For undivided roadways, the width should equal the approach roadway width.*

    b. *For bridges less than or equal to 200 feet in length, the width should be no less than the full paved width of the approach roadway when the minimum widths provided in the tables cannot be met.*

11. <u>Existing Bridges to Remain in Place</u> –

    a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

    b. *Undivided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side.*

    c. *Divided Roadways – The minimum existing bridge width should be the traveled way plus 2-foot shoulders on each side for each roadway.*

    d. *The minimum widths provided in Footnotes 11b and 11c do not include other existing features, such as auxiliary lanes, sidewalks, and bicycle lanes.*

    e. *For existing bridges that are in compliance with the applicable widths in footnotes 11b, 11c, and 11d, consideration should be given to widening the bridges to comply with the new construction width, especially in areas with high traffic volumes.*

    f. *See Section 14-2.06.05 for information on sidewalks at existing bridges.*

12. <u>Right of Way Border Width</u> – *Right of way border widths should be determined by level of development, property values, etc.*

13. <u>Roadside Clear Zones</u> –

    a. <u>With Curb</u> – *Desirably, the clear zone for open shoulder rural sections (Section 9-2.0) should also be provided for curbed sections. The 1.5-foot lateral offset should only be used in constrained areas. The distance is measured from the face of the curb to the obstruction, regardless of shoulder width.*

    b. <u>Without Curb</u> – *The clear zone for open shoulder rural sections (Section 9-2.0) should be provided. Where the recommended clear zone distances cannot be met, a 12-foot lateral offset (measured from edge of traveled way) should be provided on the outside of horizontal curves, and an 8-foot lateral offset should be provided elsewhere.*

14. <u>Slopes</u> – *If high volume change (HVC) soil is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

15. <u>Intersection Sight Distance (ISD)</u> – *The values provided in Table 14-2-H assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways.*

16. <u>Minimum Grades</u> –

    a. *On roadways with curb and gutter, the minimum longitudinal gradient should be 0.2%; however, a grade of 0.4% or greater is desirable to facilitate longitudinal drainage.*

b. *On roadways without curb and gutter, level gradients are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

c. *On bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

17. *Vertical Clearance (collector under)* –

   a. *The vertical clearance shown in the tables should be provided over the entire collector roadway width, including shoulders.*

   b. *The desirable clearance allows for future resurfacing with additional structure depth.*

   c. *For crossing routes going under the collector roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

   d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

18. *Vertical Clearance (collector over railroad) – The vertical clearance shown in the tables should typically be provided over the entire railroad right of way width.*

Table 14-2-J
GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL ROADWAYS
(New Construction/Reconstruction and 3R)

	DESIGN ELEMENT		Manual Section	With Curb	Without Curb
Design Controls	Design Year		14-2.01	20 Years	
	*Design Speed		14-2.02	20 mph – 30 mph	
	Control of Access		11-1.05	Control by Regulation (Type 3)	
	Level of Service Threshold		14-2.01	Desirable: C Minimum: D	
Cross-Section Elements	*Travel Lane Width **(1)**		14-2.06	11 ft	Des: 11 ft Min: 10 ft
	Outside Shoulder Width	*Usable **(2)**	14-2.06	Des: 4 ft Min: 2 ft	4 ft
		Paved		Same as Usable	See Note **(3)**
	*Cross Slope	Travel Lane **(4)**	14-2.06	2%	
		Shoulder		Shoulder Width > 4 ft: 4% Shoulder Width ≤ 4 ft: 2%	See Note **(5)**
	Auxiliary Lanes	Lane Width	14-2.07	Desirable: Same as Travel Lane Minimum: 10 ft	
		Usable Shoulder Width		Des: 4 ft Min: 1 ft	2 ft
	Parking Lane Width **(6)**		14-2.06	Desirable: 9 ft Minimum: 7 ft	
	Sidewalk Width		14-2.06	Minimum: 5 ft	
	New and Reconstructed Bridges	*Design Loading Structural Capacity	14-2.06	HL-93	
		Minimum Width		Approach Roadway Width	See Note **(7)**
	Existing Bridges to Remain in Place	*Design Loading Structural Capacity	14-2.06	See Note **(8)**	
		Minimum Width			
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)		11-1.01 14-2.06	See Note **(9)**	
	Roadside Clear Zone	Guardrail	9-2.0 14-2.06	Usable Shoulder Width	
		Lateral Offset		1.5 ft **(10a)**	See Note **(10b)**
	Slope Schedule **(11)**	Cut: Foreslope	14-2.08	+2%	3:1
		Cut: Depth of Ditch		1 ft	3 ft
		Cut: Backslope		Des: 3:1 Max: 2:1	Des: 3:1 Max: 2:1
		Fill: Height ≤ 5 ft		Des: 3:1 Max: 2:1	Des: 4:1 Max: 3:1
		Fill: Height > 5 ft			3:1

	DESIGN SPEED		Manual Section	20 mph	25 mph	30 mph
Alignment Elements	*Stopping Sight Distance		14-2.03	115 ft	155 ft	200 ft
	Intersection Sight Distance **(12)**		6-6.0	225 ft	280 ft	335 ft
	*Superelevation Rate		14-2.04	4%		
	*Minimum Horizontal Curve Radius (e_{max} = 4%)		14-2.04	86 ft	154 ft	250 ft
	*Maximum Grades	Residential	14-2.05	Desirable: 10% Maximum: 15%		
		Commercial/Industrial		Desirable: 5% Maximum: 8%		
	Minimum Grades **(13)**		14-2.05	See Note **(13)**		
	Vertical Curve (K-values)	*Crest	14-2.05	7	12	29
		Sag		17	26	49
	*Vertical Clearance (local roadway under) **(14)**	New/Reconstructed Bridges	2-8.02	Desirable: 17 ft Minimum: 16 ft		
		Existing Bridges		Desirable: 16 ft Minimum: 14.5 ft		
		Sign Truss/Ped. Bridge		19 ft		
	Vertical Clearance (local roadway over railroad) **(15)**		2-8.02	25 ft		

*For application of controlling design criteria, see Section 2-9.02.

Footnotes for Table 14-2-J

1. *Travel Lane Width* –

   a. *For industrial routes, 12-foot lane widths are desirable.*

   b. *The lane width should be increased by two feet for roadways that include angled on-street parking. See Section 14-2.06.06 for more information.*

2. *Shoulder Width* –

   a. *When using new construction/reconstruction criteria on existing roadways, the minimum 2-foot shoulder width in curbed sections should only be used when right of way or other constraints exist. For any widths less than the desirable width, the drainage design should include consideration of the shoulder width in order to minimize ponding on the roadway.*

   b. *See Section 14-2.06.01 for information on shoulder width relative to curb and gutter usage.*

3. *Shoulder Surface Type – Outside of the typical 2-foot paved portion, paved shoulders should not be provided except as approved in special conditions.*

4. *Cross Slope of Travel Lane (normal crown sections) – The typical cross slope of the traveled way in a normal crown section is 2.0%. In no case should the cross slope of an outer lane be less than that of the adjacent lane.*

5. *Cross Slope of Shoulder (normal crown sections)* –

   a. *The cross slope of the 2-foot paved portion of the shoulder in a normal crown section should match the cross slope of the adjacent travel lane (typically 2%).*

   b. *When more than the typical 2-foot paved shoulder is approved for special conditions, the cross slope of paved shoulders less than or equal to four feet should match the cross slope of the adjacent travel lane (typically 2%). The cross slope of paved shoulders greater than four feet should be 4%.*

   c. *The portion of the shoulder that is not paved should have a cross slope of 4%.*

   d. *In no case should the cross slope of the shoulder be less than that of the adjacent travel lane.*

6. *Parking Lanes – The widths shown in the tables apply only to parallel parking. Section 14-2.06.06 provides information on the design of angled parking.*

7. *New and Reconstructed Bridge Width (roadways without curbs)* –

   a. *The clear roadway width should preferably be the same as the approach roadway width, but should not be less than the widths shown below:*

ADT	*Width*
< 2000	*Traveled Way + three feet (each side)*
≥ 2000	*Approach Roadway Width*

   b. *For bridges greater than 100 feet in length on roadways with an ADT greater than or equal to 2000, a minimum shoulder width of three feet may be used on both the left and the right sides.*

8. *Existing Bridges to Remain in Place* –

   a. *Design Loading Structural Capacity should equal HS-20 for bridges designed under AASHTO's* Standard Specifications for Highway Bridges *and HL-93 for bridges designed under AASHTO's* LRFD Bridge Design Specifications*.*

   b. *The existing bridge width may remain unchanged if there is no crash pattern associated with the existing width. However, if a crash pattern exists, the bridge should be widened or reconstructed to 26 feet or the approach roadway width, whichever is greater. See Section 14-2.06.05 for information on sidewalks at existing bridges.*

9. *Right of Way Border Width – Right of way border widths should be determined by level of development, property values, etc.*

10. *Roadside Clear Zones – There is no specific clear zone width applicable to local roadways in urban areas. However, lateral offsets should be provided as described below:*

    a. *With Curb – A minimum lateral offset of 1.5 feet should be provided between the curb face and obstructions, such as utility poles, lighting poles, and fire hydrants. See Section 6-2.02.02 for more information on lateral offsets at intersections.*

    b. *Without Curb – On roadways without curb and gutter that have a shoulder width less than four feet, a 4-foot minimum lateral offset should be provided, measured from the edge of the traveled way.*

11. *Slopes – If high volume change soil (HVC) is present, slope adjustments may be included as part of the earthwork recommendation. See Chapter 13, "Pavement Design", for more information on HVC soil and earthwork recommendations.*

12. *Intersection Sight Distance (ISD) – The values provided in Table 14-2-J assume ISD for passenger cars turning onto a level 2-lane roadway. Section 6-6.0 provides information on adjustments for grades, trucks, and multilane roadways.*

13. *Minimum Grades –*

    a. *On roadways with curb and gutter, the minimum longitudinal gradient should be 0.2%; however, a grade of 0.4% or greater is desirable to facilitate longitudinal drainage.*

    b. *On roadways without curb and gutter, level gradients are acceptable on pavements that are crowned adequately to drain laterally. However, a longitudinal grade of 0.4% or more is desirable, except for the Mississippi Delta or other flat areas.*

    c. *On bridges, a minimum 0.5% longitudinal grade should be provided, where feasible.*

14. *Vertical Clearance (local roadway under) –*

    a. *The vertical clearance shown in the table should be provided over the entire local roadway width, including shoulders.*

    b. *The desirable clearance allows for future resurfacing with additional structure depth.*

    c. *For crossing routes going under the local roadway, the vertical clearance should be determined from the appropriate geometric design criteria table for the functional classification of the crossing route.*

    d. *Consideration should be given to raising the grade of an existing crossing route to be at or near the desirable vertical clearance if conditions exist that warrant such consideration (i.e., frequent hits by tall vehicles).*

15. *Vertical Clearance (local roadway over railroad) – The vertical clearance shown in the table should typically be provided over the entire railroad right of way width.*

14-3.0 REFERENCES

1. *Guide for the Development of Bicycle Facilities*, AASHTO, 2012.

2. *Traffic Engineering Handbook,* Institute of Transportation Engineers, 2016.

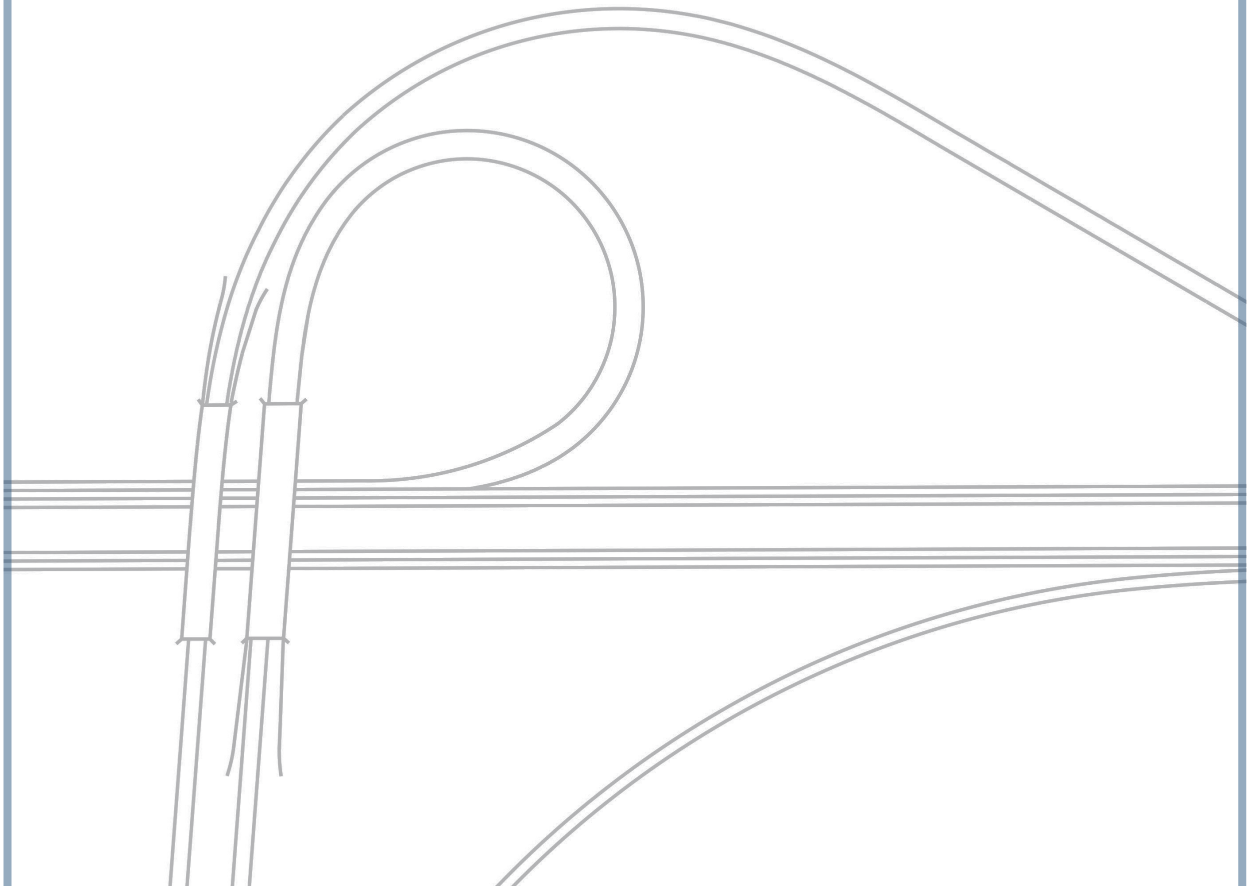
3. *A Policy on Geometric Design of Highways and Streets,* AASHTO, 2018.

4. *A Policy on Design Standards – Interstate System*, AASHTO, 2016.

5. *Access Management Manual*, Mississippi Department of Transportation, 2010.

6. *Manual on Uniform Traffic Control Devices for Streets and Highways*, National Advisory Committee on Uniform Traffic Control Devices, 2009.

7. Draft *Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way*, US Access Board, 2011.



CHAPTER 15

Roadway Design Plan Assembly

Table of Contents

Chapter 15
ROADWAY DESIGN PLAN ASSEMBLY

15-1.0 GENERAL INFORMATION

Previous chapters have provided criteria and procedures for the geometric design of a roadway facility. The appropriate design should be incorporated into the Contract Plans in a manner that can be clearly understood by contractors, material suppliers, and Department personnel assigned to supervise and inspect the construction of the project.

To ensure a consistent interpretation of the Contract Plans, individual sheets should have standard format and content, and the sequence of assembling the various types of sheets should be the same. The purpose of this chapter is to present information for uniform preparation of the Roadway Design Plans.

15-1.01 Content and Sequence

Most sets of Contract Plans contain the following types of sheets assembled in the sequence below:

1. Title Sheet
2. Detailed Index
3. Summary of Plan Revisions
4. General Notes
5. Typical Sections
6. Summary of Quantities
7. Estimated Quantities
8. Plan-Profile Sheets
9. Erosion Control Plan Sheets
10. Riparian Buffer Sheets
11. Special Design Sheets — Pavement Markings, Intersection Details, Form Grades, Construction Signing, Traffic Control Plans, etc.
12. Traffic Engineering Sheets — Permanent Signing, Traffic Signals, ITS components, etc.
13. Lighting
14. *MDOT Roadway Design Standard Drawings*
15. *MDOT Bridge Standard Drawings* (Box Culverts and Box Bridges)
16. Bridge Design Sheets (including Bridge Detailed Index, Summary of Bridge Plan Revisions, and Bridge Quantities)
17. Cross-Section Sheets

15-1.02 <u>Project Identification</u>

Each sheet in the Roadway Design Plans should be clearly identified with the project number. On all sheets in the plan set, the project number and other pertinent information should be included in the pre-assigned locations on the applicable sheet borders. Each element of the project number has a special meaning as discussed in S.O.P. No. ADM-02-08-02-000.

15-1.03 <u>Numbering System</u>

15-1.03.01 Working Numbers

Working numbers are assigned to groups of particular sheets for identification during the design process as follows:

1. <u>Title Sheet</u> – The title sheet is always sheet number 1 with no assigned working number.

2. <u>Detailed Index Sheet</u> – The first Detailed Index sheet is numbered DI-1, and any additional sheets are numbered DI-2, DI-3, etc.

3. <u>Summary of Plan Revisions Sheet</u> – The first plan revision sheet is numbered SPR-1, and any additional sheets are numbered SPR-2, SPR-3, etc.

4. <u>General Notes Sheets</u> – General Notes sheets are numbered GN-1, GN-2, etc.

5. <u>Typical Section Sheets</u> – The first typical section sheet is numbered TS-1, and any additional sheets are numbered TS-2, TS-3, etc.

6. <u>Summary of Quantities</u> – The first summary of quantities sheet is numbered SQ-1, and any additional sheets are numbered SQ-2, SQ-3, etc.

7. <u>Estimated Quantities Sheets</u> – The estimated quantities sheets are numbered EQ-1, EQ-2, etc.

8. <u>Plan-Profile Sheets</u> – The first mainline plan-profile sheet is numbered Wk. No. 3 and subsequent mainline plan-profile sheets are numbered in sequence. Additional sheets that supplement a particular plan-profile sheet (e.g., ramps, local roads) should have the same number as the mainline sheet plus an alphabetical designation in sequence (e.g., 3A). For divided roadways, separate sheets may be necessary for each roadway. In such cases, the working number should identify the roadway shown (e.g., 5LT, 5RT).

9. <u>Special Design Sheets</u> – The following list identifies the working numbers for common special design sheets:

    a. Pavement Marking Details — PMD-1, PMD-2, etc.
    b. Intersection Details — ID-1, ID-2, etc.
    c. Form Grades — FG-1, FG-2, etc.
    d. Construction Signing — CS-1, CS-2, etc.
    e. Temporary Traffic Control — TC-1, TC-2, etc.
    f. Erosion Control Plans – ECP-3, ECP-4, etc. (mainline sheets only)
    g. Riparian Buffer Plans – ECP-RB-3, ECP-RB-4, etc. (mainline sheets only)

h. Right of Way Markers Coordinates – ROW-1, ROW-2, etc.
i. Control Points – CP-1, CP-2, etc.

15-1.03.02 Sheet Numbers

Each sheet is given a number in the proper sequence. Blocks of numbers have been reserved for the various types of sheets as presented in Table 15-1-A.

Table 15-1-A
SHEET NUMBERS FOR CONTRACT PLANS

Sheet Numbers	Type of Drawing
1-1000	Title Sheet through Plan-Profile Sheets and other Special Design Roadway Sheets
1001-2000	Permanent Signs
2001-3000	Traffic Signals
3001-4000	ITS Components
4001-5000	Lighting
5001-6000	(Reserved)
6001-7000	*MDOT Roadway Design Standard Drawings*
7001-8000	*MDOT Bridge Standard Drawings* (Box Culverts and Box Bridges)
8001-9000	Bridge Design Sheets (Including Bridge Detailed Index, Summary of Bridge Plan Revisions, and Bridge Quantities)
9001 and above	Cross-Section Sheets

15-1.04 <u>Revisions</u>

Design revisions should be recorded when a revision occurs after the Final Right of Way Plans or Contract Plans have been issued.

Design changes that are made after the Final Right of Way Plans have been issued should be recorded on the Right of Way Revision sheet and include the revision issue date, revision number, revised plan sheet name, division requesting the revision, approval acknowledgment and date, and revision explanation. The revision triangle number should be placed on each plan-profile sheet corresponding to the location where changes occurred as described on the Right of Way Revision Sheet. All proposed right of way revisions should be approved by the Roadway Design Division Engineer and the Right of Way Division Administrator prior to their submittal. Before the

Contract Plans are issued, the model containing all revision dates and revision symbols should be turned off. The date that plans are issued should be recorded on the Title Sheet and on the Summary of Revisions Sheet. First order revisions, addendums, and special second order revisions should follow the same procedure for submitting Right of Way revisions, but instead using the Summary of Revisions sheet.

15-1.05 Standard Symbols

Commonly used standard symbols and abbreviations are provided in the Roadway Design CADD workspace and CADD files, which are provided in the Roadway Design Division folder of ProjectWise. The Roadway Design Division CADD Engineer should be contacted for instructions regarding access to the folder. The designer should adhere to the standard symbols shown in these documents in the preparation of all Roadway Design Plans.

15-2.0 TITLE SHEET

The title sheet serves as the cover sheet for the Contract Plans. The following information should be included on the title sheet:

1. Title Information – The external and FMS project numbers and the project location, including the route number and county where the project is located, should be recorded on the title sheet.

2. General Location Map – The general location of the project in relation to the entire state should be identified with a small star on the small-scale state map provided in the upper right corner of the sheet. Latitude and longitude at the approximate middle of the project should be recorded at the bottom of the general location map.

3. Project Layout – A small-scale layout of the entire project alignment should be shown in the center of the title sheet. The appropriate scale for a project is generally 1 inch = 2000 feet or 1 inch = 3000 feet, depending on the length of project. The minimum information that should be included on the project layout includes north arrow, township, range, section lines and numbers, urban and municipal limits, populations, county roads, highways, rivers, streams requiring bridges, railroads, airfields, cities or towns, approximate distance from either end to the next city or town, the project alignment, each plan-profile sheet and sheet number, omitted sections, bridges (including bridge numbers and site numbers), and the beginning and ending stations. The project alignment should be the predominate line of the layout. The area covered by each mainline plan-profile sheet should be sketched over the alignment, and each sheet identified by the working number. If station labels are included on the title sheet layout, they should be shown at an increment corresponding to the layout scale. For example, where a 100-foot stationing increment is typically used for plan-profile sheets at a scale of 1 inch = 100 feet, a 3000-foot stationing increment would be more suitable on a title sheet with a layout scale of 1 inch = 3000 feet.

4. General Index – In the general index block located in the upper left corner of the sheet, the applicable boxes should be checked for the plan sheets that are included in the project.

5. Bridge Locations – All proposed bridges and box bridges included in a project should be listed to the left of the project layout map. Each structure should be identified by an alphabetical designation, beginning station, total length along centerline, skew, and number and length of spans.

6. Equations – Station equations that occur along the alignment of the project should be listed under the project layout map. The list should identify the length of each equation with the appropriate sign. An algebraic summation is then made to obtain a total length of all equations.

7. Exceptions – Any omitted sections that occur along the alignment of the project should be listed as an exception under the project layout map. Bridges in place (excluding box bridges) are presented on the layout as omitted sections and are included in the length of exceptions.

8. Length Data – The project length data should be listed on the title sheet for length of roadway, length of bridges, net length of project, length of exceptions, and gross length of project.

   a. The length of roadway is the difference between the beginning and ending stations plus (or minus) the total length of the equations, minus the length of bridges and exceptions.

   b. The length of bridges is the sum of the proposed bridges along the centerline of the project.

   c. A box bridge is defined as a box culvert that has a clear distance between the inside faces of the end supports exceeding 20 feet measured along the centerline of the roadway. For separate grading and paving projects, this length is included in the length of bridges only on the grading projects. The length data for a box bridge should be measured from outside face to outside face of the end supports along the centerline of the project.

   d. The net length of project is the sum of the roadway length and bridge length.

   e. The length of exceptions is the sum of all omitted sections within the project limits.

   f. The gross length of project is the sum of the net length and the exceptions.

   As a general rule, the length data on 4-lane facilities should be computed along the centerline of the median when the median width is constant, or along the centerline of the right lane when the median width is variable. The accuracy of all computations for project length should be recorded to the nearest 0.01 foot.

9. Design Controls – The design control block consists of the design speed and design traffic data obtained from the project design data sheet (RWD-600).

10. Access Control – Projects that include a fully- or partially-controlled access facility should include a note on the title sheet declaring that roadway to be a controlled-access facility.

The type of access control should also be included in the note. See Section 11-1.05 for definitions of the different types of access control, and see the *Access Management Manual* for more information.

11. Environmental Permit Stamp – The appropriate boxes for the types of environmental permits on the project should be checked in the environmental permit stamp. The Roadway Design Environmental Engineer should approve the permit stamp prior to the Contract Plans being issued.

15-3.0 DETAILED INDEX AND GENERAL NOTES

15-3.01 Detailed Index

The second sheet of the Contract Plans normally will be the Detailed Index sheet. The Detailed Index should contain a description of each sheet in the plans, grouped under the same titles and sequence as in the general index.

On the Detailed Index sheet, the total number of sheets within the group should be recorded in parentheses immediately to the right of the title for each sheet group. The working number and sheet number should be provided adjacent to the description of each sheet (see Section 15-1.03). The latest revision date for a standard drawing, either bridge or roadway, should be provided adjacent to the sheet number.

Typical section sheets should be arranged on the Detailed Index sheet in the following sequence:

1. main facility
2. interchange ramps and loops
3. interchange local road
4. other roadways
5. frontage roads
6. county roads
7. access roads
8. detour roads

Quantity sheets should be arranged on the Detailed Index sheet in the following sequence:

1. Summary of Quantities
2. Estimated Quantities (Roadway)
3. Estimated Quantities (Permanent Signing, Traffic Signals, ITS, or Lighting)

Plan-profile sheets should be arranged in the sequence along the mainline from the beginning of the project. The working number for the first mainline sheet is 3. Where the limits of the mainline sheet contain an interchange, ramp, crossroad, on-site detour, etc., supplemental sheets (e.g., 3A) should be included to show all features of the interchange, ramp, crossroad, on-site detour, etc. The supplemental sheets should be placed immediately after the mainline sheet where they appear.

Special Design Sheets may pertain specifically to the project (e.g., traffic control sheets, intersection details, pavement marking sheets), or they may be more standard in nature and apply to any project. The project-specific sheets should precede the non-project-specific sheets in the plans.

15-3.02 General Notes

The General Notes include specific information applicable to the particular project. Information contained in the *Standard Specifications* or special provisions should not be provided as a General Note. A list of commonly used General Notes is available on the Department's web page. Designers should refer to this list for any updates to the notes prior to creating a General Notes sheet.

15-4.0 TYPICAL SECTIONS

Depending on the complexity of the project, one or more typical section sheets may be included in each set of Roadway Design Plans. The mainline typical section should be displayed first, with other typical sections following in the sequence as discussed in Section 15-3.01. Each typical section should be clearly identified by name and station limits. The following should be noted:

1. Application – Under each typical section drawing, the station limits where the typical section applies should be listed (e.g., Station 130+00 to Station 134+50).

2. Grading Projects – Details of the pavement structure are typically not available during the time that plans are being developed for the grading project. However, the estimated pavement structure thickness (above subgrade) and the proposed slope of the pavement and shoulders should be shown as a broken line.

3. Slopes – Criteria for slopes should be provided on the typical section sheets. This information can be obtained from the project design data sheet (RWD-600), which should conform to the side slope criteria presented in the tables of geometric design criteria in Chapters 2, 12, and 14.

4. Cut and Fill Sections – Typical sections should generally be drawn and labeled as half-cut section and half-fill section.

5. Special Details – Special details (e.g., sidewalks, curbs, safety edge, pavement transitions at bridge ends, slope benching) should also be included on the typical section sheets.

6. Notes – Notes that apply specifically to a typical section should be listed below the applicable drawing.

7. Traffic Data – Traffic data used to design the pavement structure should be shown in the Data for Pavement Determination box.

8. Pavement Structure – Each layer of a pavement structure should be exaggerated enough to clearly depict the various lifts. All courses should be indexed by number and accurately described on the typical section sheet as to width, thickness, and type of material. All terminology should be identical to the terminology used in the *Standard Specifications*.

15-5.0 QUANTITY SHEETS

15-5.01 Summary of Quantities

The purpose of the summary of quantities sheets is to summarize all estimated quantities by pay item.

Quantities should be segregated by county to capture the actual construction costs within the applicable county. Additionally, quantities should also be separated between urban and rural designations for the purpose of allowing greater flexibility when funding a project. Any notes that may explain how quantities were obtained or to define rates of application not contained in the *Standard Specifications* should be provided as footnotes on the summary of quantities sheets.

The same general rules apply to summary of quantities sheets provided by other Divisions (e.g., Bridge Design, Traffic Engineering).

15-5.02 Estimated Quantities

In addition to the summary of quantities sheets, roadway plans usually contain other quantity sheets entitled Estimated Quantities (Roadway). The purpose of the estimated quantities sheets is to provide tables for tabulating related quantities in order to determine a total quantity for each item. If the project includes bridges or traffic elements, similar Estimated Quantities (Bridge) or Estimated Quantities (Traffic) are included; however, estimated quantities for bridges are included with the Bridge Design Sheets, while all other estimated quantities sheets are included with the Roadway Design plans.

The Roadway Design sheet(s) generally contain columns for pipe culverts, box culverts, box bridges, ditch treatment, guardrail, curb and gutter, riprap, or other items that appear within the plans. The bridge sheet(s) generally consolidates all information by individual structure and by components for various pay items. A standardized format should be used for each sheet, as feasible. Estimated quantities should be segregated by county and by urban and rural areas.

15-6.0 PLAN–PROFILE SHEETS

The plan-profile sheets present the vertical and horizontal alignments, topography, right of way, and other details necessary for the construction of the project. All plan-profile sheets, excluding ECP sheets, should be issued in color as set forth in the Roadway Design CADD workspace.

The basic survey information, including the horizontal alignment, curve data, topography, bench marks, groundline profile, and groundline elevations, should be recorded first on the plan-profile sheets. The following information should also be noted on the plan-profile sheets:

1. Scales – Scales for rural and urban projects should be 1 inch = 50 feet horizontally and 1 inch = 5 feet vertically. Larger scales may be considered where appropriate. The horizontal scale should be designated in the plan view.

2. Elevations – Existing and proposed groundline elevations are shown along the bottom of the plan-profile sheet. For paving plans, only the plan grade elevations are presented and not the groundline elevations. The groundline profile is shown as a broken line.

3. Earthwork – Cut and fill earthwork volumes should appear above the plan grade elevations and below the original ground profile on lines denoting the cut and fill limits. Guidelines for earthwork calculations and sheet totals are discussed in Section 5-2.03.

4. Notes – Notes on the plan-profile sheets should be brief, clear, and consistent.

15-7.0 OTHER PLAN SHEETS

15-7.01 Interchange Layout Sheets

A layout sheet should be provided for all proposed interchanges to provide a plan view showing the entire interchange. The following information should be included:

1. horizontal alignment and curve data
2. proposed right of way
3. access controls
4. bridges
5. drainage structures

Contour lines may also be included to assist in understanding the drainage patterns. The interchange layout sheet should be inserted immediately after the mainline plan-profile sheet that includes the intersection station of the mainline and the crossroad.

15-7.02 Special Design Sheets

Special design sheets are typically used to present design details that cannot be described adequately on the plan-profile sheets, such as pavement markings, intersection details, traffic control, right of way marker coordinates, and vegetation schedules. Additional sheets may be included, as applicable, for form grades, non-standard superelevation transitions, and clearing limits adjacent to streams. Each special design sheet with a drawing of the subject roadway requires appropriate north arrow and scale symbol. The applicable GPS data used on the project should be recorded in the GPS Control Notes box on the Right of Way Marker Coordinates sheet.

15-7.03 Form Grades

The purpose of preparing form grades is to graphically determine the profiles of the edge of pavement at intersecting roadways, such as interchange ramps and channelized intersections. The scale normally used is 1 inch = 30 feet.

The objective is to establish a smooth profile for both edges of the ramp pavement at 25-foot intervals and to provide means for graphically determining the grade at any location.

Form grades are used to:

1. determine location of construction joints or crossover crown line
2. determine pavement slopes and slope transitions
3. accurately scale pavement width at any location
4. check rate of cross slope and maximum algebraic difference
5. check for proper pavement drainage

The plan view should clearly present:

1. elevations at 25-foot intervals along the control line of the ramp
2. pavement slope line, perpendicular (radially) to the ramp control line
3. pavement slope (percent), recorded above the pavement slope line
4. pavement width, recorded below the pavement slope line

15-8.0 CROSS-SECTION SHEETS

Cross-section sheets are the last group of sheets in the Contract Plans. The first cross-section sheet is sheet number 9001.

Mainline cross-section sheets should be sequentially numbered from beginning to end without interruption. Any supplemental cross sections (ramps, frontage roads, local roads, channel changes, etc.) should be grouped after the mainline cross sections, with each sheet clearly identified. Detailed information for preparing cross-section sheets are discussed in Chapter 5, “3D Models, Cross Sections, and Earthwork.”