SDDOT LOCAL ROADS PLAN

Chapter 2

Rural and Urban Design Criteria

RURAL AND URBAN DESIGN CRITERIA

Design Standards for Restoration, Rehabilitation, and Reconstruction

- Refer to Chapter 2 of the SDDOT Road Design Manual.

Roadway Classifications

As per AASHTO guidance, Local Project Design Standards are based on the road's functional classification. This applies to both hydraulic and roadway design.

Design criteria will be in accordance with the most current edition of the AASHTO publication, "A Policy on Geometric Design of Highways and Streets," referred to in this document as the 'AASHTO Green Book.'

The best possible design should be selected considering safety, existing and future needs, economy, reasonable maintenance costs and available funding. In restricted areas, or where there are other unusual considerations, it may not be possible to meet all minimum design values. Exceptions to applicable design criteria will be considered upon request by the county or city on a project by project basis when in the public interest and subject to approval by the SDDOT.

Local Roads & Streets

Projects not on the Federal-aid System will be designed to meet the criteria found in Chapter 5 of the AASHTO Green Book, 'Local Roads and Streets' (Appendix 1). Projects administered by the SDDOT on these roads will have a prefix of BRO.

Collector Roads & Streets

Projects on the Federal-aid System under the jurisdiction of the counties will be designed to meet the criteria found in Chapter 6 of the AASHTO Green Book, 'Collector Roads and Streets' (Appendix 2). Projects on the Federal-aid System under the jurisdiction of the cities will be designed to meet the criteria found in Chapter 6, 'Collector Roads and Streets', and in Chapter 7, 'Rural and Urban Arterials' (Appendix 3). Projects administered by the SDDOT on these roads will have a prefix of BRF.

Local Streets in Urban Areas – Appendix 1, 5.3
Collectors in Urban Areas – Appendix 2, 6.3

Design Speed

Design speed is a selected speed used to determine the various geometric design features of the roadway. The selected design speed should be a logical one with respect to the anticipated operating speed, topography, adjacent land use, modal mix and the functional classification of the roadway.

On lower-speed facilities, use of above-minimum design criteria may encourage travel at speeds higher than the design speed. The selected design speed should be consistent with the speeds that drivers are likely to travel on a given roadway. Where a reason for limiting speed is obvious, drivers are more apt to accept lower speed operation than where there is no apparent reason. A roadway of higher functional classification may justify a higher design speed than a lesser classified facility in similar topography. A low design speed, however, should not be selected where the topography is such that drivers are likely to travel at high speeds. Drivers do not adjust their speeds to the importance of the roadway, but to their perception of the physical limitations of the highway and its traffic.

Design speed is dependent on the judgement of the designer considering local conditions. A thorough discussion of design speed can be found in Chapter 2 of the AASHTO Green Book. Minimum design speeds based on functional classification, terrain, and design volume can be found as noted below.

Local Rural Roads – Appendix 1, 5.2.1.1
Rural Collectors – Appendix 2, 6.2.1.1
Local Urban Streets – Appendix 1, 5.3.1.1
Urban Collectors – Appendix 2, 6.3.1.1
Urban Arterials – Appendix 3, 7.3.2.1

Design Traffic Volume

Local Rural Roads – Appendix 1, 5.2.1.2
Rural Collectors – Appendix 2, 6.2.1.2
Local Urban Streets – Appendix 1, 5.3.1.2
Urban Collectors – Appendix 2, 6.3.1.2
Urban Arterials – Appendix 3, 7.3.2.2

Levels of Service

Local Rural Roads – Appendix 1, 5.2.1.3
Rural Collectors – Appendix 2, 6.2.1.3
Local Urban Streets – Appendix 1, 5.3.1.3
Urban Collectors – Appendix 2, 6.3.1.3
Urban Arterials – Appendix 3, 7.3.2.3

Alignment

Local Rural Roads – Appendix 1, 5.2.1.4
Rural Collectors – Appendix 2, 6.2.1.4
Local Urban Streets – Appendix 1, 5.3.1.4
Urban Collectors – Appendix 2, 6.3.1.4
Urban Arterials – Appendix 3, 7.3.2.5

Grades

Local Rural Roads – Appendix 1, 5.2.1.5
Rural Collectors – Appendix 2, 6.2.1.5
Local Urban Streets – Appendix 1, 5.3.1.5
Urban Collectors – Appendix 2, 6.3.1.5
Urban Arterials – Appendix 3, 7.3.2.6

Cross Slope

Local Rural Roads - Appendix 1, 5.2.1.6

Pavement cross slope should be adequate to provide proper drainage. Normally, cross slopes range from 1.5 to 2 percent for high-type pavements and 2 to 6 percent for low-type pavements.

High-type pavements are those that retain smooth riding qualities and good non-skid properties in all weather with little maintenance.

For low-type pavements such as surface treatments, stabilized or loose gravel, or stabilized earth surfaces, a 4 percent cross slope is desirable according to the USDOT FHWA "*Gravel Roads Construction & Maintenance Guide*". For further information on pavement cross slope, see Chapter 4 (of the AASHTO Green Book).

Rural Collectors – Appendix 2, 6.2.1.6

Pavement cross slope should be adequate to provide proper drainage. Normally, cross slopes range from 1.5 to 2 percent for high-type pavements. High-type pavements are those that retain smooth riding qualities and good non-skid properties in all weather under heavy traffic volumes and loadings with little maintenance required.

Low-type pavements are those with treated earth surfaces and those with loose aggregate surfaces. A cross slope of 4 to 6 percent is desirable for low-type pavements. For further information, see the section on "Cross Slope" in Chapter 4 (of the AASHTO Green Book).

(Additional Guidance: USDOT FHWA "*Gravel Roads Construction & Maintenance Guide*")

Local Urban Streets – Appendix 1, 5.3.1.6
Urban Collectors – Appendix 2, 6.3.1.6
Urban Arterials – Appendix 3, 7.3.2.8

Note for Cross Slope with Bridges on Gravel Roads

This topic needs to be discussed at each location as the difference between the deck slope and the road slope can have quite an impact on the length of taper sections between a bridge and the existing road. 4 percent is nationally recommended for gravel road cross slopes and SDDOT requires all hard surfaces (bridge decks & asphalt/concrete pavements over boxes) to be 2 percent.

Superelevation

Super elevation rates will be according to current SDDOT standards. As stated in the SDDOT Road Design Guide, because of South Dakota's weather conditions, the maximum permissible rate of super elevation is 6 percent. This will apply to all paved surface roads. The maximum permissible rate of super elevation on gravel surface roads will be 8 percent. If other conditions arise that warrant consideration of greater rates, these will be discussed on an individual basis.

Local Rural Roads – Appendix 1, 5.2.1.6

For rural roads with paved surfaces, super elevation should be not more than 12 percent except where snow and ice conditions prevail, in which case the super elevation should be not more than 8 percent. For aggregate roads, super elevation should be not more than 12 percent.

Super elevation runoff is the length of highway needed to accomplish the change in cross slope from a section with the adverse crown removed to a fully super elevation section. Minimum lengths of runoff are given in Chapter 3 *(of the* AASHTO Green Book*)*. Adjustments in design runoff lengths may be desirable for smooth riding, surface drainage, and good appearance. For a general discussion on this topic, see Chapter 3 (of the AASHTO Green Book).

Rural Collectors – Appendix 2, 6.2.1.7

Many rural collector highways have curvilinear alignments. A super elevation rate compatible with the design speed should be used. For rural collectors, super elevation should not exceed 12 percent. Where snow and ice conditions may be a factor, the super elevation rate should not exceed 8 percent. Super elevation runoff denotes the length of highway needed to accomplish the change in cross slope from a section with the adverse crown removed to a fully super elevation section and vice versa. Adjustments in design runoff lengths may be needed to provide a smooth ride, surface drainage, and good appearance. The section on "Horizontal Alignment" in Chapter 3 (of the AASHTO Green Book) provides a detailed discussion on super elevation for appropriate design speeds.

Local Urban Streets – Appendix 1, 5.3.1.7
Urban Collectors – Appendix 2, 6.3.1.7
Urban Arterials – Appendix 3, 7.3.2.7

Sight Distance

Local Rural Roads – Appendix 1, 5.2.1.6
Rural Collectors – Appendix 2, 6.2.1.8
Local Urban Streets – Appendix 1, 5.3.1.8
Urban Collectors – Appendix 2, 6.3.1.8
Urban Arterials – Appendix 3, 7.3.2.4

Roadway Width

As South Dakota is primarily an agricultural state, a minimum subgrade width of 32' has long been the standard for all local rural and rural collector roads constructed with state and federal funding through the SDDOT LGA Office. This provides for a desired top width of 28', comprised of 2-12' driving lanes and 2-2' shoulders on paved surfaces. Over the years counties have adopted this standard in addition to even wider shoulders as can be found in their transportation plans as funded by the SDDOT State Planning & Research Program for Local Governments. The lane and shoulder widths noted above are also consistent with the minimum standard for all state rural highways as outlined in Chapter 7 of the Road Design Manual entitled "Cross Sections" providing a continuity for the agricultural transportation needs of South Dakota. Deviations from this standard can be made based on local needs.

Local Rural Roads – Appendix 1, 5.2.2.1
Rural Collectors – Appendix 2, 6.2.2.1
Local Urban Streets – Appendix 1, 5.3.2.1
Urban Collectors – Appendix 2, 6.3.2.1
Urban Arterials – Appendix 3, 7.3.3.1

Number of Lanes

Local Rural Roads – Appendix 1, 5.2.2.2
Rural Collectors – Appendix 2, 6.2.2.2
Local Urban Streets – Appendix 1, 5.3.2.2
Urban Collectors – Appendix 2, 6.3.2.2
Urban Arterials – Appendix 3, 7.3.3.4

Parking Lanes

Rural Collectors – Appendix 2, 6.2.2.3
Local Urban Streets – Appendix 1, 5.3.2.3
Urban Collectors – Appendix 2, 6.3.2.3
Urban Arterials – Appendix 3, 7.3.3.7

Medians

Local Rural Roads – Appendix 1, 5.2.2.4
Rural Collectors – Appendix 2, 6.2.2.4
Local Urban Streets – Appendix 1, 5.3.2.4
Urban Collectors – Appendix 2, 6.3.2.4
Urban Arterials – Appendix 3, 7.3.3.5

Curbs

Local Urban Streets – Appendix 1, 5.3.2.5
Urban Collectors – Appendix 2, 6.3.2.5
Urban Arterials – Appendix 3, 7.3.3.3

Right-of-Way Width

Local Rural Roads – Appendix 1, 5.2.2.3
Rural Collectors – Appendix 2, 6.2.2.5
Local Urban Streets – Appendix 1, 5.3.2.6
Urban Collectors – Appendix 2, 6.3.2.6
Urban Arterials – Appendix 3, 7.3.3.9

Provision for Utilities

Local Urban Streets – Appendix 1, 5.3.2.7
Urban Collectors – Appendix 2, 6.3.2.7
Urban Arterials – Appendix 3, 7.3.10

Utility Adjustments

Adjustment of utilities will be in accordance with South Dakota State Law and 23 CFR 645. The county or city is responsible for utility notification and coordinating any utility relocation work. Assistance can be requested of the Utility Coordinator of the SDDOT Road Design Office.

Utility facilities will be adjusted or removed from the right-of-way in cases where they constitute a safety hazard. Minimum lateral clearances, as noted in the Rural & Urban Design Criteria section of the Local Roads Plan, as applicable, may be allowed on a project by project basis considering traffic volume, right-of-way width, removal cost and location. Exceptions to these criteria shall be approved by the Administration Program Manager.

Prior to advertising of contracts, the FHWA Division Administrator will be furnished a Utility Certification on all projects to assure compliance with applicable provisions.

Border Area

Local Urban Streets – Appendix 1, 5.3.2.8
Urban Collectors – Appendix 2, 6.3.2.8
Urban Arterials – Appendix 3, 7.3.3.8

Bicycle and Pedestrian Facilities

Local Rural Roads – Appendix 1, 5.2.2.5
Rural Collectors – Appendix 2, 6.2.2.6
Local Urban Streets – Appendix 1, 5.3.2.9
Urban Collectors – Appendix 2, 6.3.2.9
Urban Arterials – Appendix 3, 7.3.9

Cul-de-Sacs and Turnarounds

Local Urban Streets – Appendix 1, 5.3.2.10

Alleys

Local Urban Streets – Appendix 1, 5.3.2.11

Driveways

Local Rural Roads – Appendix 1, 5.2.2.6
Local Urban Streets – Appendix 1, 5.3.2.12
Urban Collectors – Appendix 2, 6.3.2.10

Structures – New and Reconstructed

Local Rural Roads – Appendix 1, 5.2.2.7.1

Rural Collectors – Appendix 2, 6.2.3.1
Local Urban Streets – Appendix 1, 5.3.3.1
Urban Collectors – Appendix 2, 6.3.3.1
Urban Arterials – Appendix 3, 7.3.5.1

Structures – Vertical Clearance

Local Rural Roads – Appendix 1, 5.2.2.7.2
Rural Collectors – Appendix 2, 6.2.3.2
Local Urban Streets – Appendix 1, 5.3.3.2
Urban Collectors – Appendix 2, 6.3.3.2
Urban Arterials – Appendix 3, 7.3.5.2

Structure Widths

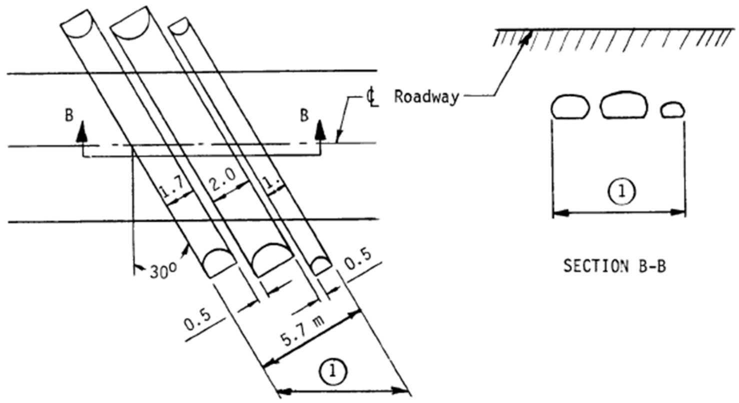
Due to South Dakota being an agriculture state, all structures should accommodate at least a 28' roadway top as discussed in the previous section on Roadway Widths. For bridges the 28' clear width is measured between the insides of the bridge rail. If approaching roadway is wider than 28' (lane & shoulder), the bridge clear width shall match the overall width of the approaching roadway. Box culverts should be long enough to accommodate 2-12' lanes plus the clear zone to the inside face of the parapet. This has been the standard practice for decades. Deviations from this practice can be made based on local needs which shall be documented in the TS&L letter. Consideration for future widening of the roadway should be part of the structure width determination, or future widening may result in the structure being too narrow (bridge) or within the clear zone (box). This is not a desirable situation so thinking ahead and providing a structure width that will cover the future is important. If a master transportation plan or local standard has been adopted that differs from the recommendations above, those standards should be discussed in determining the width requirements to be used for design.

Structure Design Considerations – Appendix 4

Structure Definitions - National Bridge Inspection Standards (NBIS) and South Dakota Codified Law (SDCL)

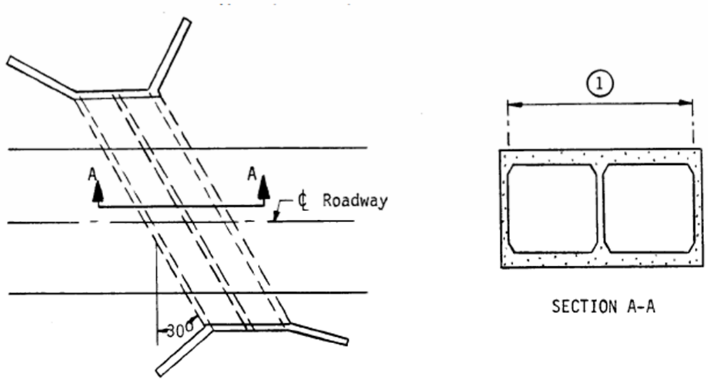
The NBIS definition of a bridge is as follows: A structure including supports, erected over a depression or an obstruction, as water, highway, or railway, the structure having a length measured along the center of the roadway of more than twenty feet between undercopings of abutments or extreme ends of openings for multiple boxes and pipes where the clear distance between openings is less than half of the smaller contiguous (within a sequence) opening. Refer to the figure below.

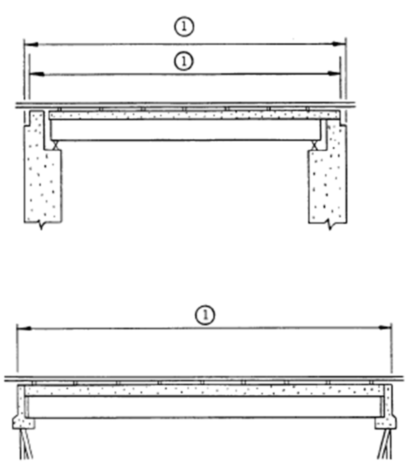
Figure II-1

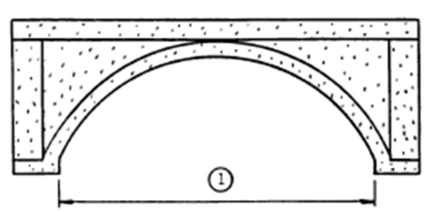


Multiple pipes may be considered a bridge if the distance between the pipes is less than half of the smallest opening and the structure length is greater than 6.1 meters (20 feet). In the above illustration, the structure length is recorded as 6.58 meters:

$$\text{Structure Length} = \frac{5.7\text{ m}}{\cos 30^{\circ}} = 6.58\text{ m}$$







(1) Item 49 - Structure Length

According to South Dakota Codified Law (SDCL) a structure is a culvert when it cannot be classified as a bridge and provides an opening under a roadway. (SDCL 31-14-1) Culverts shall be no less than 24 feet in length. (SDCL 31-12-18)

If an option is included for a replacement structure less than the 20 feet in length, the local government needs to be made aware that this will remove that structure from future federal and state funding. If this is the chosen option and it is on a Township Road, SDDOT will require a Joint Powers Agreement between the County and the Township prior to the project being let to bid.

Bridge Rail

Steel rail shall meet NCHRP 350 Test Level 2 or better. In South Dakota the rail used is T101, T115, or SL1. Plates are provided by LGA. Turned down ends are used if the ADT is less than 150. Approach rail is needed if ADT is greater than 150 which is covered by either the SL1 or T101 with approach rail details available through LGA.

Concrete rail shall meet 32" MASH TL-3 and can be a Jersey, sloped, or vertical shape. Anything taller will adversely affect agricultural traffic and a higher test level will increase the cost of construction due to the extra steel required and possibly a thicker deck.

Rail designs that may be needed for special conditions (i.e. an approach or intersection located within such close proximity to a structure as to interfere with the standard rail placement, rehabilitation of existing rail, etc.) will be in accordance with current SDDOT guidelines.

Box Culverts – Parapets, Aprons, Mixing of Materials

Parapets are standard on all rural box culverts at the request of the counties. The parapets are essential to reduce sluff of shoulder material reducing the long term maintenance issues. The parapet is considered a hazard, according to AASHTO design standards, and must be kept outside of the clear zone.

Aprons are most often concrete as they provide another cut off wall and make a box easier to access for inspection. Riprap is acceptable in situations where landowners do not want cattle to get through the box. It is recommended boxes are either all precast or all cast-in-place as mixing these has resulted in higher costs. This has been the standard in South Dakota on rural structures for decades, in order to keep costs down for the owners.

Hydraulics

This section addresses standards and design criteria specific to roads not on the state system. Because of the relatively low traffic volumes and extensive roadway mileage on roads functionally classified as local roads and streets (off-system) and collector roads and streets (federal aid system), design criteria are comparatively lower as a matter of practicality. Although the *South Dakota Drainage Manual* https://dot.sd.gov/doing-business/engineering/design-services/forms-manuals shall be referred to for guidance on performing drainage investigations and preparing hydraulic designs, this section shall be the primary starting point for designers working on local government projects to ensure designs are appropriate for the noted local road systems.

Hydraulic recommendations will be reported on the Hydraulic Data Sheet. (Note that a special Hydraulic Data Sheet for Local Government projects has been prepared for this use and is available for distribution in electronic format as provided by LGA).

Collector Roads and Streets / On-System

Hydraulically size the structure(s) so no roadway overtopping occurs for less than, or equal to, the 25-year frequency flood event.

Where the project ADT (current or 20-year projection) is 100 or less, the design may be reduced to a 10-year frequency provided the following is met: 1) The current structure frequency is less than a 25-year frequency; 2) there is an overtopping section located away from the structure; and 3) the local government is willing to accept this reduction in standards and service to their taxpayers along this route.

Bridge designs shall provide for a minimum of one foot of freeboard at the design event from the low bridge girder to the water surface for bridge installations with the desirable overflow section being away from the bridge location.

If current conditions do not meet the noted design frequencies and the local government has no issue with the current level of service, they may opt to have the structure sized to meet the current frequency. This situation shall be documented on the hydraulic data sheet, along with local government concurrence in the form of commission action or a letter from their highway department.

Local Roads and Streets / Off-System

When an overtopping section is available away from the structure location, hydraulically size the structure so no roadway overtopping occurs for less than, or equal to, the 10-year frequency flood event.

When no overtopping section away from the structure exists, the structure must be hydraulically sized for no roadway overtopping for less than, or equal to, the 25-year frequency flood event.

Bridge designs shall provide for a minimum of one foot of freeboard at the design event from the low bridge girder to the water surface for bridge installations with the desirable overflow section being away from the bridge location.

If current conditions do not meet the noted design frequencies and the local government has no issue with the current level of service, they may opt to have the structure sized to meet the current frequency. This situation shall be documented on the hydraulic data sheet, along with local government concurrence in the form of commission action or a letter from their highway department.

Eligible & Ineligible Costs – Structure Projects

When expending funds, federal or state, for the replacement or rehabilitation of structures, every effort is made to maintain those funds for work on the structure.

Minimization of grading work has long been required for federally funded projects as grading is only eligible between what is called the "touchdown" points of the structure. These are the limits of roadway disturbance needed to remove and replace the structure. Grading outside of what is determined to be the touchdown points, is ineligible for bridge funds and will need to be funded by state or local funds. This can be found in 23 CFR 650.405 (c)

> *(c) Ineligible work. Except as otherwise prescribed by the Administrator, the costs of long approach fills, causeways, connecting roadways, interchanges, ramps, and other extensive earth structures, when constructed beyond the attainable touchdown point, are not eligible under the bridge program.*

In addition to grading outside of the touchdown points, the South Dakota Association of County Highway Superintendents has created a list of additional items that are ineligible for federal and state funded structure projects. These items include the following: right-of-way costs, utility relocations, roadway surfacing, sidewalk/bikepath concrete off the structure, drop inlets and other urban drainage features off the structure, fencing, aesthetics, and off-site environmental mitigation and monitoring costs.

Ineligible work included in a project must be clearly discussed, defined, and most importantly agreed to in writing with the local government as they will be responsible for 100 percent of the associated costs. Ineligible costs must be clearly marked in the plans.

On-Site Traffic Detours at Structures – Try to Avoid

On-site traffic detours should be avoided if at all possible as they are expensive and must be constructed as a part of the project as per all the DOT/Environmental requirements. Discussing this with the local government early is essential.

Roadside Design

Local Rural Roads – Appendix 1, 5.2.2.8

Clear Zones - Horizontal Clearance to Obstructions

Local Rural Roads – Appendix 1, 5.2.2.8.1

A clear zone of 10 ft or more from the edge of the traveled way, appropriately graded with relatively flat slopes and rounded cross-sectional design, is desirable. An exception may be made where guardrail protection is provided. The recovery area should be clear of all unyielding objects such as trees, fixed sign supports, utility poles, light poles, and any other fixed objects that might severely damage an out-of-control vehicle.

To the extent practical, where another highway or railroad passes over, the structure should be designed so that the pier or abutment supports have lateral clearance as great as the clear roadside area on the approach roadway. For further information on providing roadside lateral clearance, see the *AASHTO Roadside Design Guide (3).*

Where it is not practical to carry the full-width approach roadway across an overpass or other bridge, an appropriately transitioned roadside barrier should be provided. At selected locations, such as the outside of a sharp curve, a broader recovery area with greater horizontal clearances should be provided to any roadside obstruction.

Rural Collectors – Appendix 2, 6.3.4.1

For rural collector roads with a design speed of 45 mph or less, a minimum clear zone of 10 ft measured from the edge of the traveled way should be provided. This recovery area should be clear of all unyielding objects such as trees, fixed sign supports, utility poles, light poles, and other fixed objects. The benefits of removing these obstructions should be weighed against any environmental and aesthetic effects.

For rural collector roads with a design speed of 50 mph or more, the *AASHTO Roadside Design Guide (3)* should be used for guidance in selecting an appropriate clear-zone width.

Guidance can also be found in Chapter 10 of the SDDOT Road Design Manual.

The approach roadway width (traveled way plus shoulders) should be carried across an overpass or bridge, where practical. Approach roadside barriers, anchored to the bridge rails or parapets, should be provided. Sidewalks should extend across a bridge if the approach roadway has sidewalks or sidewalk areas. To the extent practical, where another highway or railroad passes over the roadway, the overpass structure should be designed so that the pier or abutment supports have lateral clearance as great as the clear zone on the approach roadway. Where a setback beyond the clear zone is not practical, roadside barrier protection should be provided at the piers.

Local Urban Streets – Appendix 1, 5.3.4.1
Urban Collectors – Appendix 2, 6.3.4.1
Urban Arterials – Appendix 3, 7.3.4.1

Lateral Offset

Local Rural Roads – Appendix 1, 5.2.2.8.2
Rural Collectors – Appendix 2, 6.2.4.2
Local Urban Streets – Appendix 1, 5.3.4.2
Urban Collectors – Appendix 2, 6.3.4.2
Urban Arterials – Appendix 3, 7.3.4.2

Foreslopes

Local Rural Roads – Appendix 1, 5.2.2.8.3

The maximum rate of foreslope depends on the stability of local soils as determined by soil investigation and local experience. Slopes should be as flat as practical, and other factors should be considered to determine the design slope. Flat foreslopes increase safety by providing a maneuver area in emergencies, are more stable than steep slopes, aid in the establishment of plant growth, and simplify maintenance work. Vehicles that leave the traveled way can often be kept under control if slopes are gentle and drainage ditches are well-rounded. Such recovery areas should be provided where terrain and right-of-way controls permit.

Combinations of rate and height of slope should provide for vehicle recovery. Where controlling conditions (such as high fills, right-of-way restrictions, or the presence of rocks, watercourses, or other roadside features) make this impractical, consideration should be given to the provision of guardrail, in which case the maximum rate of foreslope could be used.

Cut sections should be designed with adequate ditches. Preferably, the foreslope should not be steeper than 1V:2H, and the ditch bottom and slopes should be well-rounded. The backslope should not exceed the maximum required for stability.

Rural Collectors – Appendix 2, 6.2.4.3

The maximum rate of foreslope should depend on the stability of local soils as determined by soil investigation and local experience. Slopes should be as flat as practical, taking into consideration other design constraints. Flat foreslopes improve safety by providing a maneuvering area in emergencies, are more stable than steep slopes, aid in the establishment of plant growth, and simplify maintenance work. Roadside barriers may be used where topography and right-of-way are restrictive and a need is justified.

Drivers who inadvertently leave the traveled way can often recover control of their vehicles if foreslopes are 1V:4H or flatter and shoulders and ditches are well rounded or otherwise made traversable. Such recoverable slopes should be provided where terrain and right-of-way conditions allow.

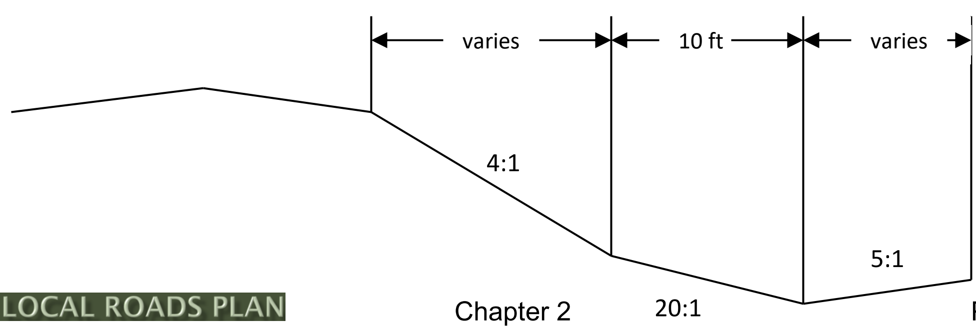
Where provision of recoverable slopes is not practical, the combinations of rate and height of slope provided should be such that occupants of an out-of-control vehicle have a good chance of survival. Where high fills, right-of-way restrictions, watercourses, or other problems render such designs impractical, roadside barriers should be considered, in which case the maximum rate of fill slope may be used. Reference should be made to the current edition of the *AASHTO Roadside Design Guide (3)*. For further information, see the section on “Traffic Barriers” in Chapter 4 (of the AASHTO Green Book).

Cut sections should be designed with adequate ditches. Preferably, the foreslope should not be steeper than 1V:3H and, where practical, should be 1V:4H or flatter. The ditch bottom and slopes should be well rounded, and the backslope should not exceed the maximum needed for stability.

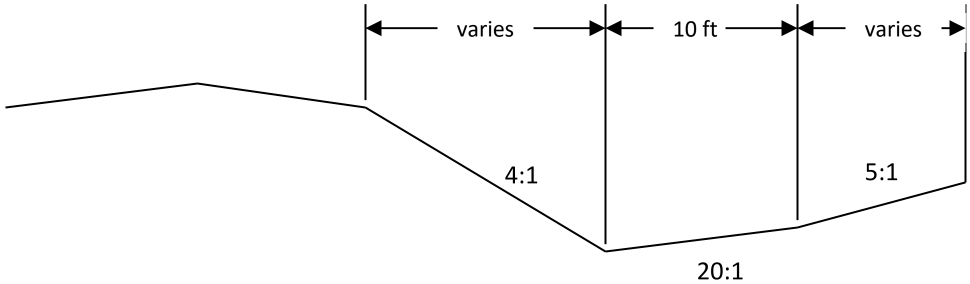
Typical Cross Section – All Rural Roads

The typical section will include a crown slope of 4 percent for gravel surfaces or 2 percent for paved surfaces, 4:1 inslopes, a standard 10' ditch at 20:1, and 5:1 backslopes. When conditions don’t allow this, the inslope (foreslope) can be steepened to 1V:2H on Local Rural Roads or 1V:3H on Rural Collectors as discussed in the previous section on Foreslopes.

The following typical section keeps the ditch drainage farther from the roadway but requires larger work limits and potentially the need for more right-of-way.



The following typical section works well in areas where the project limits and impact to the right-of-way must be kept to a minimum.



Intersection Design

Local Rural Roads – Appendix 1, 5.2.2.9
Rural Collectors – Appendix 2, 6.2.5
Local Urban Streets – Appendix 1, 5.3.5
Urban Collectors – Appendix 2, 6.3.5
Urban Arterials – Appendix 3, 7.3.11

Railroad-Highway Grade Crossings

Local Rural Roads – Appendix 1, 5.2.2.10
Rural Collectors – Appendix 2, 6.2.6
Local Urban Streets – Appendix 1, 5.3.6
Urban Collectors – Appendix 2, 6.3.6
Urban Arterials – Appendix 3, 7.3.7

Traffic Control Devices

Local Rural Roads – Appendix 1, 5.2.2.11
Rural Collectors – Appendix 2, 6.2.7
Local Urban Streets – Appendix 1, 5.3.7
Urban Collectors – Appendix 2, 6.3.7

Roadway Lighting

Local Urban Streets – Appendix 1, 5.3.8
Urban Collectors – Appendix 2, 6.3.8
Urban Arterials – Appendix 3, 7.3.18

Drainage

Local Rural Roads – Appendix 1, 5.2.2.12
Rural Collectors – Appendix 2, 6.2.8
Local Urban Streets – Appendix 1, 5.3.9
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Erosion Control and Landscaping

Local Rural Roads – Appendix 1, 5.2.2.13
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Urban Collectors – Appendix 2, 6.3.10 & 6.3.11
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Speed Transitions Entering Rural Towns

Rural Collectors – Appendix 2, 6.2.10

Recreational Roads – Appendix 1, 5.4

Resource Recovery and Local Service Roads – Appendix 1, 5.5

Low-Volume Roads – Local and Minor Collector Roads with Traffic Volumes of 2,000 Vehicles Per Day or Less

Low-volume roads often present a unique challenge because low traffic volumes and reduced frequency of crashes make designs normally applied on higher volume roads less cost effective. AASHTO has produced a publication to assist in these situations which can be found in Appendix 5, AASHTO Second Edition of the 2019 “Guidelines for Geometric Design of Low-Volume Roads”.

Chapter 2 – Appendix 1

AASHTO *"A Policy on Geometric Design of Highways and Streets"*

(2018, 7th Edition, 2nd Printing)

Chapter 5: Local Roads and Streets

5 Local Roads and Streets

5.1 INTRODUCTION

This chapter presents guidance on the application of geometric design criteria to facilities functionally classified as local roads and streets. The chapter is subdivided into sections on rural, urban, and special-purpose local roads.

A local road or street serves primarily to provide access to farms, residences, businesses, or other abutting properties. Although local roads and streets may be planned, constructed, and operated with the predominant function of providing access to adjacent property for a variety of users, some local roads and streets serve a limited amount of through traffic. On these roads, the through traffic is local in nature and extent rather than regional, intrastate, or Interstate. Such roads properly include geometric design and traffic control features more typical of collectors and arterials.

Local roads and streets constitute a high proportion of the roadway mileage in the United States. The traffic volume generated by the abutting land uses are largely short trips or a relatively small part of longer trips where the local road connects with major streets or highways of higher classifications. Because of the relatively low traffic volumes and the extensive roadway mileage, design criteria for local roads and streets are of a comparatively low order as a matter of practicality. However, to provide traffic mobility and safety—together with the essential economy in construction, maintenance, and operation—they should be planned, located, and designed to be suitable for predictable traffic operations and should be consistent with the development and culture abutting the right-of-way.

In constrained or unusual conditions, it may not be practical to meet the design criteria presented in this chapter. In such cases, the goal should be to obtain the best practical alignment, grade, sight distance, and drainage that are consistent with terrain, development (present and anticipated), crash reduction, and available funds.

Drainage, both on the pavement itself and from the sides and subsurface, is an important design consideration. Inadequate drainage can lead to high maintenance costs and adverse operational conditions. In areas of substantial snowfall, roadways should be designed so that there is sufficient storage space outside the traveled way for plowed snow and proper drainage for melting conditions.

It may not be cost-effective to design local roads and streets that carry less than 2,000 vehicles per day using the same criteria applicable to higher volume roads or to make extensive improvements to such very low-volume roads. Alternate design criteria may be considered for local and minor collector roads and streets that carry 2,000 vehicles per day or less in accordance with the AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads* (*1*).

The specific dimensional design criteria presented in this chapter are appropriate as a guide for new construction of local roads and streets. Projects to improve existing roads differ from new construction in that the performance of the existing road is known and can guide the design process. Features of the existing design that are performing well may remain in place, while features that are performing poorly should be improved, where practical. Chapter 1 presents a flexible, performance-based design process that can be applied in developing projects on collector roads and streets.

5.2 LOCAL ROADS IN RURAL AREAS

This section presents guidance on the design of local roads and streets in the rural and rural town contexts. The primary differences between geometric design in the rural and rural town contexts are in the choice of design speed and the increased need in the rural town context to provide parking, to serve increased pedestrian and bicyclist flows, and blend in with the community.

5.2.1 General Design Considerations

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

5.2.1.1 Design Speed

Design speed is a selected speed used to determine the various design features of the roadway. Geometric design features should be appropriate for environmental and terrain conditions and consistent with the selected design speed. Designers are encouraged to select design speeds equal to or greater than the minimum values shown in Table 5-1. Low design speeds are generally applicable to roads with winding alignment in rolling or mountainous terrain or where environmental conditions dictate. High design speeds are generally applicable to roads in level terrain or where other environmental conditions are favorable. Intermediate design speeds would be appropriate where terrain and other environmental conditions are a combination of those described for low and high speed. Table 5-1 lists values for minimum design speeds as appropriate for traffic volumes and types of terrain; terrain types are discussed further in Chapters 2 and 3.

Table 5-1. Minimum Design Speeds for Local Roads in Rural Areas

Type of Terrain	U.S. Customary: Design Speed (mph) for Specified Design Volume (veh/day)					Metric: Design Speed (km/h) for Specified Design Volume (veh/day)				
	under 50	50 to 250	250 to 400	400 to 2,000	2,000 and over	under 50	50 to 250	250 to 400	400 to 2,000	2,000 and over
Level	30	30	40	50	50	50	50	60	80	80
Rolling	20	30	30	40	40	30	50	50	60	60
Mountainous	20	20	20	30	30	30	30	30	50	50

5.2.1.2 Design Traffic Volume

Roads should be designed for a specific traffic volume and a desired level of service. The average daily traffic (ADT) volume, either for the current year, the projected opening year, or projected to some future design year, should be the basis for design. Usually, the design year is about 20 years into the future, but may range from the current year to 20 years depending on the nature of the improvement.

5.2.1.3 Levels of Service

Procedures for estimating the traffic operational performance of particular highway designs are presented in the *Highway Capacity Manual* (HCM) (*17*), which also presents a thorough discussion of the level-of-service concept. Although the choice of an appropriate design level of service is left to the highway agency, designers should strive to provide the highest level of service practical and consistent with anticipated conditions. Level-of-service characteristics are discussed in Section 2.4.5 and summarized in Table 2-2.

5.2.1.4 Alignment

Alignment between control points should be designed to be as favorable as practical, consistent with the environmental impact, topography, terrain, design traffic volume, and the amount of reasonably obtainable right-of-way. Sudden changes between curves of widely different radii or between long tangents and sharp curves should be avoided. Where practical, the design should include passing opportunities. Where crest vertical curves and horizontal curves occur together, greater-than-minimum sight distance should be provided so that the horizontal curves are visible to approaching drivers.

5.2.1.5 Grades

Suggested maximum grades for local roads in rural areas are shown in Table 5-2 as a function of type of terrain and design speed.

Table 5-2. Maximum Grades for Local Roads in Rural Areas

Type of Terrain	U.S. Customary										Metric								
	Maximum Grade (%) for Specified Design Speed (mph)										Maximum Grade (%) for Specified Design Speed (km/h)								
	15	20	25	30	35	40	45	50	55	60	20	30	40	50	60	70	80	90	100
Level	9	8	7	7	7	7	7	6	6	5	9	8	7	7	7	7	6	6	5
Rolling	12	11	11	10	10	10	9	8	7	6	12	11	11	10	10	9	8	7	6
Mountainous	17	16	15	14	14	13	12	10	10	—	17	16	15	14	13	12	10	10	—

NOTE: Short lengths of grade in rural areas, such as grades less than 500 ft [150 m] in length, one-way downgrades, and grades on low-volume roads (AADT less than 2,000 veh/day) may be up to 2 percent steeper than the grades shown in this table.

5.2.1.6 Cross Slope

Traveled-way cross slope should be adequate to provide proper drainage. Normally, cross slopes range from 1.5 to 2 percent for paved surfaces and 2 to 6 percent for unpaved surfaces.

For unpaved surfaces, such as stabilized or loose gravel, and for stabilized earth surfaces, a cross slope of at least 3 percent is desirable. For further information on pavement and shoulder cross slopes, see Sections 4.2.2 and 4.4.3.

Superelevation—For roads in rural areas with paved surfaces, superelevation should be not more than 12 percent, except where snow and ice conditions prevail, in which case the superelevation should be not more than 8 percent. For unpaved roads, superelevation should be not more than 12 percent.

Superelevation runoff is the length of roadway needed to accomplish a change in outside-lane cross slope from zero (flat) to full superelevation, or vice versa. Minimum lengths of runoff are presented in Section 3.3.8.2. Adjustments in design runoff lengths may be desirable for smooth riding, surface drainage, and good appearance. For a general discussion on this topic, see Section 3.3.8, "Transition Design Controls."

Sight Distance—Minimum stopping sight distance and passing sight distance should be as shown in Tables 5-3 and 5-4. Criteria for measuring sight distance, both vertical and horizontal, are as follows: for stopping sight distance, the height of eye is 3.5 ft [1.08 m] and the height of object is 2.00 ft [0.60 m]; for passing sight distance, the height of eye remains the same, but the height of object is 3.50 ft [1.08 m]. Section 3.2 provides a general discussion of sight distance.

Table 5-3. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves

U.S. Customary			
Initial Speed (mph)	Design Stopping Sight Distance (ft)	Rate of Vertical Curvature, K[a] (ft/%)	
		Crest	Sag
15	80	3	10
20	115	7	17
25	155	12	26
30	200	19	37
35	250	29	49
40	305	44	64
45	360	61	79
50	425	84	96
55	495	114	115
60	570	151	136
65	645	193	157

Metric			
Initial Speed (km/h)	Design Stopping Sight Distance (m)	Rate of Vertical Curvature, K[a] (m/%)	
		Crest	Sag
20	20	1	3
30	35	2	6
40	50	4	9
50	65	7	13
60	85	11	18
70	105	17	23
80	130	26	30
90	160	39	38
100	185	52	45

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Sections 3.2.2 and 3.4.6 for details.)

Table 5-4. Design Controls for Crest Vertical Curves Based on Passing Sight Distance

U.S. Customary		
Design Speed (mph)	Design Passing Sight Distance (ft)	Rate of Vertical Curvature, K[a] (ft/%)
20	400	57
25	450	72
30	500	89
35	550	108
40	600	129
45	700	175
50	800	229
55	900	289
60	1000	357
65	1100	432

Metric		
Design Speed (km/h)	Design Passing Sight Distance (m)	Rate of Vertical Curvature, K[a] (m/%)
30	120	17
40	140	23
50	160	30
60	180	38
70	210	51
80	245	69
90	280	91
100	320	119

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Sections 3.2.4 and 3.4.6 for details.)

5.2.2 Cross-Sectional Elements

5.2.2.1 Width of Roadway

The minimum roadway width is the sum of the traveled way and graded shoulder widths given in Table 5-5. Graded shoulder width is measured from the edge of the traveled way to the point of intersection of shoulder slope and foreslope. Where roadside barriers are proposed, it is desirable to provide a minimum offset of 4.0 ft [1.2 m] from the traveled way to the barrier whenever practical. For further information, see Section 4.4, "Shoulders" and Section 4.10.2, "Longitudinal Barriers." For information on roadway widening to accommodate vehicle off-tracking, see "Derivation of Design Values for Widening on Horizontal Curves," Section 3.3.9.1.

Where bicycle facilities are included as part of or adjacent to the roadway, refer to AASHTO's *Guide for the Development of Bicycle Facilities* (*6*). Where pedestrian facilities are provided, they must be accessible to and usable by individuals with disabilities (*19, 21*); consult the AASHTO *Guide for Planning, Design, and Operation of Pedestrian Facilities* (*2*) and the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*18*) for design elements not addressed in References 21 and 19.

5.2.2.2 Number of Lanes

Two travel lanes usually can accommodate the normal traffic volume on local roads in rural areas. If exceptional traffic volumes occur in specific areas, additional lanes may be provided based on an operational analysis. Provisions for climbing and passing lanes are covered in Section 3.4, "Vertical Alignment."

Table 5-5. Minimum Width of Traveled Way and Shoulders for Two-Lane Local Roads in Rural Areas

U.S. Customary			
Design Speed (mph)	Minimum Width of Traveled Way (ft) for Specified Design Volume (veh/day)		
	under 400	400 to 2000	over 2000
15	18	20[a]	22
20	18	20[a]	22
25	18	20[a]	22
30	18	20[a]	22
35	18	20[a]	22
40	18	20[a]	22
45	20	22	22
50	20	22	22
55	22	22	22b
60	22	22	22b
65	22	22	22b
All speeds	**Width of graded shoulder on each side of the road (ft)**		
	2	3	6

Metric			
Design Speed (km/h)	Minimum Width of Traveled Way (m) for Specified Design Volume (veh/day)		
	under 400	400 to 2000	over 2000
20	5.4	6.0[a]	6.6
30	5.4	6.0[a]	6.6
40	5.4	6.0[a]	6.6
50	5.4	6.0[a]	6.6
60	5.4	6.0[a]	6.6
70	6.0	6.6	6.6
80	6.0	6.6	6.6
90	6.6	6.6	6.6[b]
100	6.6	6.6	6.6[b]
All speeds	**Width of graded shoulder on each side of the road (m)**		
	0.6	1.0	1.8

[a] For roads in mountainous terrain with design volume of 400 to 600 veh/day, an 18-ft [5.4-m] traveled-way width may be used.

[b] Consider using traveled-way width of 24 ft [7.2 m] where substantial truck volumes are present or agricultural equipment frequently uses the road

5.2.2.3 Right-of-Way Width

Providing right-of-way widths that accommodate construction, adequate drainage, and proper maintenance of a highway is a very important part of the overall design. Wide rights-of-way permit the construction of gentle slopes, resulting in reduced crash severity potential and providing for easier and more economical maintenance. The procurement of sufficient right-of-way at the time of the initial construction permits the widening of the roadway and the widening and strengthening of the pavement at a reasonable cost as traffic volumes increase.

In developed areas, it may be necessary to limit the right-of-way width. However, the right-of-way width should not be less than that needed to accommodate all the elements of the design cross sections, utilities, and appropriate border areas.

5.2.2.4 Medians

Medians are generally not provided for local roads in rural areas. For additional information on medians, see Section 5.3, "Local Streets in Urban Areas."

5.2.2.5 Bicycle and Pedestrian Facilities

Many local roadways are sufficient to accommodate bicycle traffic. Where dedicated facilities for bicycles are desired, they should be in accordance with AASHTO's *Guide for the Development of Bicycle Facilities* (*6*).

Sidewalks are not normally found along local roads in rural areas. However, for areas where the designer desires to accommodate pedestrians, sidewalks should generally have a width of at least 5 ft [1.5.m]. Sidewalks with a 4-ft [1.2 m] width may be provided, but passing areas at least 5 ft [1.5 m] in width and 5 ft [1.5 m] in length must be provided at least every 200 ft [60 m]. Where curbs are present, curb ramps must be provided at crosswalks to accommodate persons with disabilities. All pedestrian facilities must be accessible to and usable by individuals with disabilities (*19, 21*). Additional design guidance can be found in Section 4.17.1, "Sidewalks," in the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*2*), and in the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*18*).

5.2.2.6 Driveways

A driveway is an access constructed within a public right-of-way, connecting a public roadway with adjacent property and intended to provide vehicular access.

Some of the principles of intersection design apply directly to driveways. In particular, driveways should have well-defined locations. Large graded or paved areas adjacent to the traveled way that allow drivers to enter or leave the street randomly should be discouraged.

Sight distance is an important design control for driveways. Driveway locations where sight distance is limited should be avoided. Vertical obstructions to essential sight distances should be controlled by regulations. Driveway regulations should address width of entrance, spacing, and placement with respect to property lines and intersecting streets, angle of entry, and vertical alignment and pedestrian accessibility where driveways cross sidewalks. Driveways should be situated as far away from intersections as practical, particularly if the driveway is located near an arterial street.

Flared driveways are preferred because they are distinct from intersection delineations and can properly handle turning movements. Design guidance related to driveway elements including grade, width, channelization, cross slope, and other geometrics is presented in Section 4.15.2 and in the *Guide for the Geometric Design of Driveways* (*14*). Further guidance on the design of sidewalk-driveway interfaces can be found in AASHTO's *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*2*) and the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*18*).

5.2.2.7 Structures

5.2.2.7.1 New and Reconstructed Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the current *AASHTO LRFD Bridge Design Specifications* (*9*). Except as otherwise indicated in this chapter and in Chapter 4, the dimensional design of structures should also be in accordance with Reference 9.

The minimum design loading for new bridges on local roads in rural areas should be the HL-93 design vehicle live loads.

The minimum clear roadway widths for new and reconstructed bridges should be as given in Table 5-6. For general discussion of structure widths, see Chapter 10.

Table 5-6. Minimum Clear Roadway Widths and Design Loadings for New and Reconstructed Bridges

U.S. Customary			Metric		
Design Volume (veh/day)	Minimum Clear Roadway Width for Bridges[a]	Design Loading Structural Capacity	Design Volume (veh/day)	Minimum Clear Roadway Width for Bridges[a]	Design Loading Structural Capacity
under 400	Traveled way + 2 ft (each side)	HL-93	under 400	Traveled way + 0.6 m (each side)	HL-93
400 to 2,000	Traveled way + 3 ft (each side)	HL-93	400 to 2,000	Traveled way + 1.0 m (each side)	HL-93
over 2,000	Approach roadway width[b]	HL-93	over 2,000	Approach roadway width[b]	HL-93

[a] Where the approach roadway width (traveled way plus shoulders) is surfaced, that surface width should be carried across the structures.

[b] For bridges in excess of 100 ft [30 m] in length, the minimum width of traveled way plus 3 ft [1 m] on each side is acceptable.

5.2.2.7.2 Vertical Clearance

Vertical clearance at underpasses should be at least 14 ft [4.3 m] over the entire roadway width, with an allowance for future resurfacing. The vertical clearance to sign supports and to bicycle and pedestrian overpasses should be 1.0 ft [0.3 m] greater than the highway structure clearance.

5.2.2.8 Roadside Design

Roadside design has an important role in reducing the severity of crashes that may occur when vehicles run off the road. It may not be practical to provide an obstacle-free roadside on local roads and streets. However, every effort should be made to provide as much clear roadside as is practical. This becomes more important as speeds increase. The judicious use of guardrail and flatter slopes helps to reduce crash severity for vehicles that leave the roadway. There are typically two primary considerations for roadside design along the traveled way for local roads in rural areas—clear zone and lateral offset. Foreslope is another important consideration in roadside design with regard to both crash reduction and slope stability.

5.2.2.8.1 Clear Zones

A clear zone of 7 to 10 ft [2 to 3 m] or more from the edge of the traveled way, appropriately graded with relatively flat slopes and rounded cross-sectional design, is desirable. An exception may be made where guardrail protection is provided. The clear zone should be clear of all unyielding objects such as trees, sign supports, utility poles, light poles, and any other fixed objects that might increase the potential severity of a crash when a vehicle runs off the road. Further guidance on clear zones can be found in the AASHTO *Roadside Design Guide* (*5*).

A source of alternative clear zone design criteria that may be considered for local roads and streets that carry 2,000 vehicle per day or less is the AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads* (*1*).

5.2.2.8.2 Lateral Offset

Lateral offset is defined in Section 4.6.2. Further discussion and suggested guidance on the application of lateral offsets is provided in the AASHTO *Roadside Design Guide* (*5*).

The full approach width (traveled way plus shoulders) should be carried along the roadway and across bridges and overpasses where practical. To the extent practical, where another highway or railroad passes over the roadway, the overpass should be designed so that the pier or abutment supports, including barrier protection systems, have a lateral offset equal to or greater than the lateral offset on the approach roadway.

On facilities without a curb and where shoulders are present, the AASHTO *Roadside Design Guide* (*5*) provides suggested guidance concerning the provision of lateral offsets.

5.2.2.8.3 Foreslopes

The maximum rate of foreslope depends on the stability of local soils as determined by soil investigation and local experience. Slopes should be as flat as practical, taking into consideration other design constraints. Flat foreslopes reduce potential crash severity for vehicles that run off the road by providing a maneuver area in emergencies. In addition, they are more stable than steep slopes, aid in the establishment of plant growth, and simplify maintenance work. Vehicles that leave the traveled way can often be kept under control if slopes are gentle and drainage ditches are well-rounded. Such recovery areas should be provided where terrain and right-of-way controls permit.

Combinations of rate and height of slope should provide for vehicle recovery. Where controlling conditions (such as high fills, right-of-way restrictions, or the presence of rocks, watercourses, or other roadside features) make this impractical, consideration should be given to the provision of guardrail, in which case the maximum rate of foreslope consistent with slope stability may be used.

Cut sections should be designed with adequate ditches. Preferably, the foreslope should not be steeper than 1V:2H, and the ditch bottom and slopes should be well-rounded. The backslope should not exceed the maximum rate needed for stability.

5.2.2.9 Intersection Design

Intersections should be carefully located to avoid steep profile grades and to provide adequate approach sight distance. An intersection should not be situated just beyond a short-crest vertical curve or on a sharp horizontal curve. When there is no practical alternate to locating an intersection on a curve, the approach sight distance on each leg should be checked, and where practical, backslopes should be flattened and horizontal or vertical curves lengthened to provide additional sight distance. The driver of a vehicle approaching an intersection should have an unobstructed view of the entire intersection and sufficient lengths of the intersecting roadways to permit the driver to anticipate and avoid potential collisions. Sight distances at intersections with six different types of traffic control are presented in Section 9.5, "Intersection Sight Distance."

Intersections should be designed with corner radii adequate for a selected design vehicle, representing a larger vehicle that is anticipated to use the intersection with some frequency. For information on minimum turning radius, see Section 9.6, "Turning Roadways and Channelization." Where turning volumes are significant, auxiliary lanes and channelization should be considered.

Intersection legs that operate under stop control should intersect at right angles, wherever practical, and should not intersect at an angle less than 75 degrees. For more information on intersection angle, see Section 9.4.2, "Alignment."

5.2.2.10 Railroad–Highway Grade Crossings

Appropriate grade-crossing warning devices should be installed at railroad–highway grade crossings on local roads and streets. Details of the devices to be used are given in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*12*). In some states, the final approval of these devices may be vested in an agency having oversight over railroads.

Sight distance is an important consideration at railroad–highway grade crossings. There should be sufficient sight distance along the road and along the railroad tracks for an approaching driver to recognize the crossing, perceive the warning device, determine whether a train is approaching, and stop if necessary. If crossing gates are not provided, adequate sight distance along the track is also needed for drivers of stopped vehicles to decide when it is safe to proceed across the tracks. For further information on railroad–highway grade crossings, see Section 9.12.

The roadway width at all railroad crossings should be the same as the width of the approach roadway. Crossings that are located on bicycle routes that are not perpendicular to the railroad may need additional paved shoulder for bicycles to maneuver over the crossing. For further information, see the AASHTO *Guide for the Development of Bicycle Facilities* (*6*).

5.2.2.11 Traffic Control Devices

Signs, pavement and other markings, and, where appropriate, traffic signal controls are essential elements for all local roads and streets. Refer to the MUTCD (*12*) for details of the devices to be used and, for some conditions, warrants for their use.

5.2.2.12 Drainage

Drainage, both on the pavement and from the sides and subsurface, is an important design consideration. Inadequate drainage can lead to high maintenance costs and adverse operational conditions. In areas of significant snowfall, roadways should be designed so that there is sufficient storage space outside the traveled way for plowed snow and proper drainage for melting conditions. Further guidance can be found in the *AASHTO Drainage Manual* (*7*).

5.2.2.13 Erosion Control and Landscaping

Consideration should be given to the preservation of the natural groundcover and the growth of shrubs and trees within the right-of-way when designing local roads in rural areas. Shrubs, trees, and other vegetation should be considered in assessing the sight distance available to the driver and the lateral offset to roadside objects. Seeding, mulching, sodding, or other acceptable measures for covering slopes, swales, or other erodible areas should be considered in the local road design in rural areas.

For further information about erosion control and landscaping, see Section 3.6.1, "Erosion Control and Landscape Development."

5.2.2.14 Design of Local Streets in the Rural Town Context

The design of local streets in the rural town context is similar to the design of local streets in the suburban and urban contexts, which is addressed in Section 5.3.

5.3 LOCAL STREETS IN URBAN AREAS

This section presents guidance on the design of local streets in urban areas. Local streets in urban areas are designed with a flexible approach to meet the needs of the suburban, urban, and urban core contexts. Local streets generally have lower traffic volumes than collectors and arterials and lower speeds are appropriate because the emphasis is on serving the adjacent developments. A flexible and balanced design approach to serve all transportation modes appropriately should be applied. The balance among transportation modes may differ between projects based on the demand flows for each transportation mode and established neighborhood plans. The design guidance given below should be adapted to the context and needs of each individual neighborhood and street.

5.3.1 General Design Considerations

A local street in an urban area is a public roadway that serves motor vehicles, transit, pedestrians, and bicyclists. The street includes the entire area within the right-of-way and usually accommodates public utility facilities within the right-of-way. The development or improvement of streets should be based on a functional street classification that is part of a comprehensive community development plan. The design criteria should be appropriate for the ultimately planned development.

Local streets in urban areas fall within three functional classifications: arterials, collectors, and local access routes, which are discussed in Chapter 1. Geometric design guidance is provided for collector streets in Chapter 6 and for arterial streets in Chapter 7. This chapter does not present a complete discussion of all design criteria that apply to local streets. However, where there are substantial differences from the criteria used in design of other functional classes, specific design guidance is given below.

The design features of local streets in urban areas are constrained by practical limitations to a greater extent than those of similar roads in rural areas. The two major design controls are:

- the type and extent of urban development, which often limit the available right-of-way, and
- zoning or regulatory restrictions.

Some streets serve primarily to provide access to adjacent residential development areas. In such cases, the overriding consideration is to foster a community environment whereas the convenience of the motorist is secondary. Other local streets not only provide access to adjacent development but also serve limited through traffic. Traffic operational performance may be an important concern on such streets.

On streets serving industrial or commercial areas, the vehicle dimensions, traffic volumes, and vehicle loads differ greatly from those on residential streets, and different dimensional and structural design values are appropriate. The major design controls for such streets are intended to provide efficient operations. Where a particular design feature varies depending on the area served (e.g. residential, commercial, or industrial), different design guidelines are presented for each condition. The designer should be apprised of local ordinances and resolutions that affect certain design features.

5.3.1.1 Design Speed

Design speed is not a major factor for local streets in urban areas because in the typical street grid, the closely spaced intersections usually limit vehicular speeds. For consistency in design elements, design speeds ranging from 20 to 30 mph [30 to 50 km/h] may be used, depending on available right-of-way, terrain, anticipated use by pedestrians and bicyclists, adjacent development, and other area controls. Since the function of local streets is to provide access to adjacent property, all design elements should be consistent with the character of activity on and adjacent to the street, and should encourage speeds generally not exceeding 30 mph [50 km/h].

5.3.1.2 Design Traffic Volume

Traffic volume is not usually a major factor in determining the geometric criteria to be used in designing residential streets. Traditionally, such streets are designed with a standard two-lane cross section, but a four-lane cross section may be appropriate in certain urban areas, as governed by traffic volume, administrative policy, or other community considerations.

Traffic volume is a major factor for streets serving industrial or commercial areas. The ADT projected to some future design year should be the design basis. It usually is difficult and costly to modify the geometric design of an existing street unless provision is made at the time of initial construction. Design traffic volumes in such areas should be forecast for at least 10 years, and preferably 20 years, into the future.

5.3.1.3 Levels of Service

Procedures for estimating the traffic operational level of service for particular highway designs are presented in the *Highway Capacity Manual* (HCM) (*17*), which also presents a thorough discussion of the level-of-service concept. Although the choice of an appropriate design level of service is left to the highway agency, designers should provide the highest level of service practical and consistent with the project context. Level-of-service characteristics are discussed in Section 2.4.5 and summarized in Table 2-2.

5.3.1.4 Alignment

Alignment in residential areas should closely fit with the existing topography to minimize the need for cuts or fills. The function of local streets in residential areas is to provide land access, and therefore these streets should be designed to discourage through traffic. Street alignment in commercial and industrial areas should be commensurate with the topography but should be as direct as practical.

The minimum radius for horizontal curves should be the greater of 100 ft [30 m] or the minimum radius for the applicable design speed shown in Table 3-7.

5.3.1.5 Grades

Grades for local residential streets should be as level as practical, consistent with the surrounding terrain. Grades for local residential streets should be less than 15 percent. Where grades of 4 percent or steeper are needed, the drainage design may become critical. On such grades, special care should be taken to prevent erosion on slopes and open drainage facilities.

Streets in commercial and industrial areas should have grades less than 8 percent, and flatter grades should be encouraged.

To provide for proper drainage, the desirable minimum grade for streets with outer curbs should be 0.50 percent, but a minimum grade of 0.30 percent may be used. Further guidance can be found in the *AASHTO Drainage Manual* (*7*)

5.3.1.6 Cross Slope

Pavement cross slope should be sufficient to provide proper drainage. Normally cross slopes range from 1.5 to 2 percent for paved surfaces and 2 to 6 percent for unpaved surfaces where there are flush shoulders. Where there are outer curbs, cross slopes steeper than the guidelines given above by about 0.5 to 1 percent are desirable for the lane adjacent to the curb.

For unpaved surfaces, such as stabilized or loose gravel or stabilized earth surfaces, a cross slope of at least 3 percent is desirable. For further information on pavement cross slope, see Section 4.2.2.

Where shoulders are intended to be used as pedestrian facilities, the shoulder must be accessible to and usable by individuals with disabilities (*19, 21*). For additional guidance, refer to the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*18*).

5.3.1.7 Superelevation

Superelevation on horizontal curves may be advantageous for local street traffic operations in specific locations, but in built-up areas the combination of wide pavement areas, proximity of adjacent development, control of cross slope, profile for drainage, frequency of cross streets, and other urban features often combine to make the use of superelevation impractical or undesirable. Therefore, superelevation usually is not provided on local streets in residential and commercial areas; it may be considered on local streets in industrial areas to facilitate operation.

If superelevation is used, horizontal curves should be designed for a maximum superelevation rate of 4 percent. If terrain dictates sharp curvature, a maximum superelevation rate of 6 percent may be justified if the curve is long enough to provide an adequate superelevation transition. Minimum lengths of superelevation runoff and a detailed discussion of superelevation are found in Section 3.3.8.2.

5.3.1.8 Sight Distance

Minimum stopping sight distance for local streets should range from 100 to 200 ft [30 to 60 m] depending on the design speed (see Table 3-1). Design for passing sight distance seldom is applicable on local streets.

5.3.2 Cross-Sectional Elements

5.3.2.1 Width of Traveled Way

Lanes for moving traffic preferably should be 10 to 11 ft [3.0 to 3.3 m] wide, and in industrial areas they should be 12 ft [3.6 m] wide. Where the available or attainable width of right-of-way imposes severe limitations, 9-ft [2.7-m] lanes can be used in residential areas, and 11-ft [3.3-m] lanes can be used in industrial areas. Added turning lanes where used at intersections should be at least 9 ft [2.7 m] wide, and desirably 10 to 12 ft [3.0 to 3.6 m] wide, depending on the percentage of trucks.

Where bicycle facilities are included as part of the design, refer to the AASHTO *Guide for the Development of Bicycle Facilities* (*6*).

5.3.2.2 Number of Lanes

On residential streets where the primary function of the street is to provide access to adjacent development and foster a community environment, at least one unobstructed moving lane must be provided even where parking occurs on both sides. The level of user inconvenience occasioned by the lack of two moving lanes is remarkably low in areas where single-family units prevail. Local residential street patterns are such that travel distances are less than 0.5 mi [1 km] from the trip origin to a collector street. In multifamily-unit residential areas, a minimum of two moving traffic lanes to accommodate opposing traffic may be desirable. In many residential areas, a minimum roadway width of 26 ft [8 m] is needed where on-street parking is permitted. This curb face-to-curb face width of 26 ft [8 m] provides a 12-ft [3.6-m] center travel lane that provides for the passage of fire trucks and two 7-ft [2.2-m] parking lanes. Opposing conflicting traffic will yield and pause in the parking lane area until there is sufficient width to pass.

In commercial areas where there are midblock left turns, it may be advantageous to provide an additional continuous two-way left-turn lane in the center of the roadway.

5.3.2.3 Parking Lanes

Where used in residential areas, a parallel parking lane at least 7 ft [2.1 m] wide should be provided on one or both sides of the street, as appropriate to the conditions of lot size and intensity of development. In commercial and industrial areas, parking lane widths should be at least 8 ft [2.4 m] and are usually provided on both sides of the street.

Parking lane width determination in commercial and industrial areas should consider use of the parking lane for moving traffic during peak periods where industries have high employment concentrations. Where curb and gutter sections are used, the gutter pan width should be considered as part of the parking lane width. Where on-street parking spaces are designated, a portion of spaces should be accessible to persons with disabilities. For more details refer to the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*18*).

5.3.2.4 Medians

Local streets in urban areas often do not have medians. However, where medians are provided on local streets in urban areas, they are primarily to enhance the environment and to act as buffer strips. These buffer strips should be designed to minimize interference with access to the land abutting the roadway. A discussion of the various median types appears in Section 4.11.

5.3.2.5 Curbs

Local streets in urban areas normally are designed with curbs to allow greater use of available width and for control of drainage, protection of pedestrians, and delineation. The curb should be 4 to 6 in. [100 to 150 mm] high, depending on drainage considerations and traffic control.

On divided streets, the type of median curbs used should be compatible with the width of the median and the type of turning movement control.

Vertical curbs with heights of 6 in. [150 mm] or more adjacent to the traveled way should be offset at least 1 ft [0.3 m]. Where a curb-and-gutter section is provided, the gutter pan width should be used as the offset distance. For additional information regarding curbs, see Section 4.7.

5.3.2.6 Right-of-Way Width

The right-of-way width should be sufficient to accommodate the ultimate planned roadway including median (if used), shoulder (if used), landscaping strip, sidewalks, bicycle facilities, on-street parking, utility strips in the border areas, and outer slopes.

5.3.2.7 Provision for Utilities

In addition to the primary purpose of serving vehicular traffic and in accordance with state law or municipal ordinance, streets also often accommodate public utility facilities within the street right-of-way. Use of the rights-of-way by utilities should be planned to minimize interference with traffic using the street. Utilities must be located such that they do not make pedestrian facilities inaccessible. References 3 and 10 provide general principles for location and construction of utilities to minimize conflict between the use of the street right-of-way for vehicular movement and for its secondary purpose of providing space for location of utilities.

5.3.2.8 Border Area

A border area should be provided along streets to reduce the potential for collisions involving motorists and pedestrians as well as for aesthetic reasons. The street alignment should be selected to minimize roadside slopes. However, the preservation and enhancement of the environment is important in the design and construction of local streets.

The border area between the roadway and the right-of-way line should be wide enough to serve several purposes, including serving as a buffer space between pedestrians and vehicular traffic, sidewalk space, snow storage, an area for placement of underground and aboveground utilities, and an area for maintainable aesthetic features such as grass or other landscaping. A border area of 10 ft [3.0 m] or wider is desirable.

Where the available right-of-way is limited and in areas of high right-of-way costs, a border width of 2 ft [0.6 m] may be tolerated where there is no sidewalk.

5.3.2.9 Bicycle and Pedestrian Facilities

Local roadways and streets are generally sufficient to accommodate bicycle traffic. However, where dedicated facilities are desired, they should be planned and designed in accordance with the AASHTO *Guide for the Development of Bicycle Facilities* (*6*) or the FHWA *Separated Bike Lane Planning and Design Guide* (*13*).

Sidewalks used for pedestrian access to schools, parks, shopping areas, and transit stops and sidewalks in commercial areas should be provided along both sides of the street, where practical. Additional design guidance can be found in Section 4.17.1, "Sidewalks," and further guidance on designing for transit can be found in the AASHTO *Guide for Geometric Design of Transit Facilities on Highways and Streets* (*8*). In residential areas, sidewalks should be provided on at least one side of all local streets and are desirable on both sides of the street. The sidewalks should be located as far as practical from the traveled way and are usually close to the right-of-way lines.

Sidewalk widths of 5 ft [1.5 m] should generally be provided. The minimum sidewalk width is 4 ft [1.2 m]; where sidewalk widths are less than 5 ft [1.5 m], passing areas at least 5 ft [1.5 m] in width must be provided at least every 200 ft [60 m]. Sidewalk widths of 8 ft [2.4 m] or greater may be needed in commercial areas. If roadside appurtenances are situated on the sidewalk adjacent to the curb, additional width may be needed to secure the clear width. Greater sidewalk widths should be considered for higher volume sidewalks and where the sidewalk is against the curb or wall.

Curb ramps must be provided at crosswalks to accommodate persons with disabilities. Further discussion of curb ramps appears in Section 4.17.3. Where pedestrian facilities are provided, they must be accessible to and usable by individuals with disabilities (*19, 21*); consult the

AASHTO *Guide for Planning, Design, and Operation of Pedestrian Facilities* (*2*) and the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*18*) for design elements not addressed in References 19 and 21.

Transit facilities are not generally provided on local streets, but where transit facilities are provided, design guidance can be found in the AASHTO *Guide for Geometric Design of Transit Facilities on Highways and Streets* (*8*).

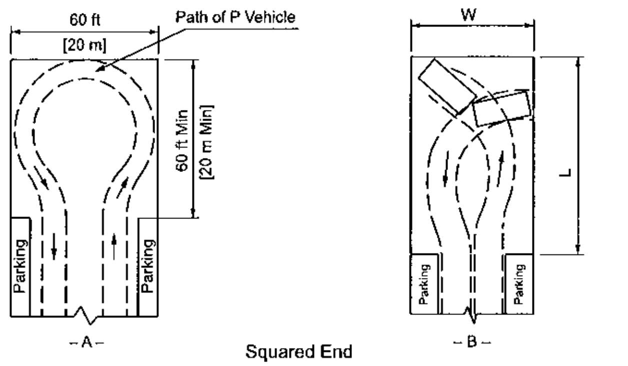
5.3.2.10 Cul-de-Sacs and Turnarounds

A local street open at one end only should have a special turning area at the closed end. This turning area desirably should be circular and have a radius appropriate to the vehicle types expected. Minimum outside radii of 30 ft [10 m] in residential areas and 50 ft [15 m] in commercial and industrial areas are commonly used.

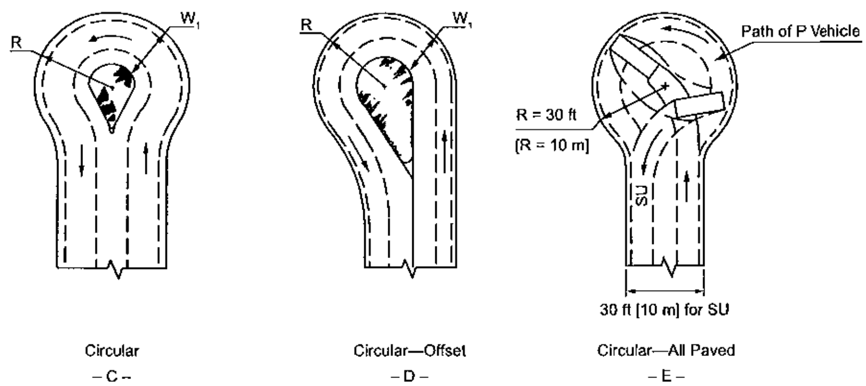
A dead-end street narrower than 40 ft [12 m] usually should be widened to enable passenger vehicles, and preferably delivery trucks, to make U-turns or at least turn around by backing only once. The design commonly used is a circular pavement symmetrical about the centerline of the street sometimes with a central island, as shown in Figure 5-1C, which also shows minimum dimensions for the design vehicles. Although this type of cul-de-sac operates satisfactorily, improved operations may be obtained if the design is offset so that the entrance-half of the pavement is in line with the approach-half of the street, as shown in Figure 5-1D. One steering reversal is avoided on this design. Where a radius of less than 50 ft [15 m] is used, the island should be bordered by sloping curbs to permit the maneuvering of an occasional oversized vehicle.

An all-paved plan, as opposed to an island configuration, with a 30-ft [10-m] outer radius, shown in Figure 5-1E, needs little additional paving. If the approach pavement is at least 30 ft [10 m] wide, the result is a cul-de-sac on which passenger vehicles can make the customary U-turn and SU design trucks can turn by backing only once.

Other variations or shapes of cul-de-sacs that include right-of-way and site controls may be provided to permit vehicles to turn around by backing only once. Several types (Figures 5-1F, 5-1G, 5-1H, and 5-1I) may also be suitable for alleys. The geometry of a cul-de-sac should be altered if adjoining residences also use the area for parking.



Design Vehicle	U.S. Customary [ft]		Metric (m)	
	W	L	W	L
P	30	60	[10]	[20]
SU	50	100	[15]	[30]



Design Vehicle	U.S. Customary [ft]		Metric (m)	
	R	W	R	W
P	30	18	[10]	[6]
WB-40 [WB-12]	42	25	[13]	[8]
WB-50 [SU & WB-15]	47	30	[15]	[10]

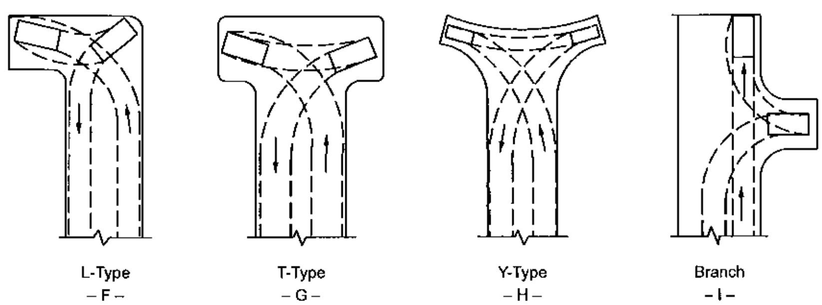


Figure 5-1. Types of Cul-de-Sacs and Dead-End Streets

5.3.2.11 Alleys

Alleys provide access to the side or rear of individual land parcels. They are characterized by a narrow right-of-way and range in width from 16 to 20 ft [5 to 6 m] in residential areas and up to 30 ft [10 m] in industrial areas.

Alleys should be aligned parallel to, or concentric with, the street property lines. It is desirable to situate alleys so that both ends of the alley are connected either to streets or to other alleys. Where two alleys intersect, a triangular corner cutoff of not less than 10 ft [3 m] along each alley property line should be provided. Dead-end alleys should include a turning area in accordance with Figure 5-2. This dead-end turning area design may be suitable for application on some very low-volume roads.

Curb return radii at street intersections may range from 5 ft [1.5 m] in residentially zoned areas to 10 ft [3 m] in industrial and commercial areas where large numbers of trucks are expected. Alleys should have grades established to meet as closely as practical the existing grades of the abutting land parcels. The longitudinal grade should not be less than 0.2 percent.

Alley cross sections may be V-shaped with transverse slopes of 2.5 percent toward a center V gutter. Runoff is thereby directed to a catch basin in the alley or to connecting street gutters. Where alleys cross sidewalks, accessibility on the sidewalk must be maintained.

5.3.2.12 Driveways

A driveway is an access constructed within a public right-of-way, connecting a public roadway with adjacent property and intended to provide vehicular access into that property in a manner that will not cause the blocking of any sidewalk, border area, or street roadway.

Some of the principles of intersection design apply directly to driveways. In particular, driveways should have well-defined locations. Large graded or paved areas adjacent to the traveled way that allow drivers to enter or leave the street randomly should be discouraged.

Sight distance is an important design control for driveways. Driveway locations where sight distance is not sufficient should be avoided. Vertical obstructions to essential sight distances should be controlled by regulations. Driveway regulations should address width of entrance, spacing, and placement with respect to property lines and intersecting streets, angle of entry, vertical alignment, and number of entrances to a single property. This will reduce the likelihood of crashes and provide maximum use of curb space for parking where permitted. Driveways should be situated as far away from intersections as practical, particularly if the driveway is located near an arterial street.

Driveway returns should not be less than 3 ft [1 m] in radius. Flared driveways are preferred because they are distinct from intersection delineations, can properly handle turning movements, and can minimize problems for persons with disabilities. Design guidance related to driveway elements including grade, width, channelization, cross slope, and other geometrics is presented in the *Guide for the Geometric Design of Driveways* (*14*). Further guidance on the design of sidewalk–driveway interfaces can be found in AASHTO's *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*2*) and the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*18*).

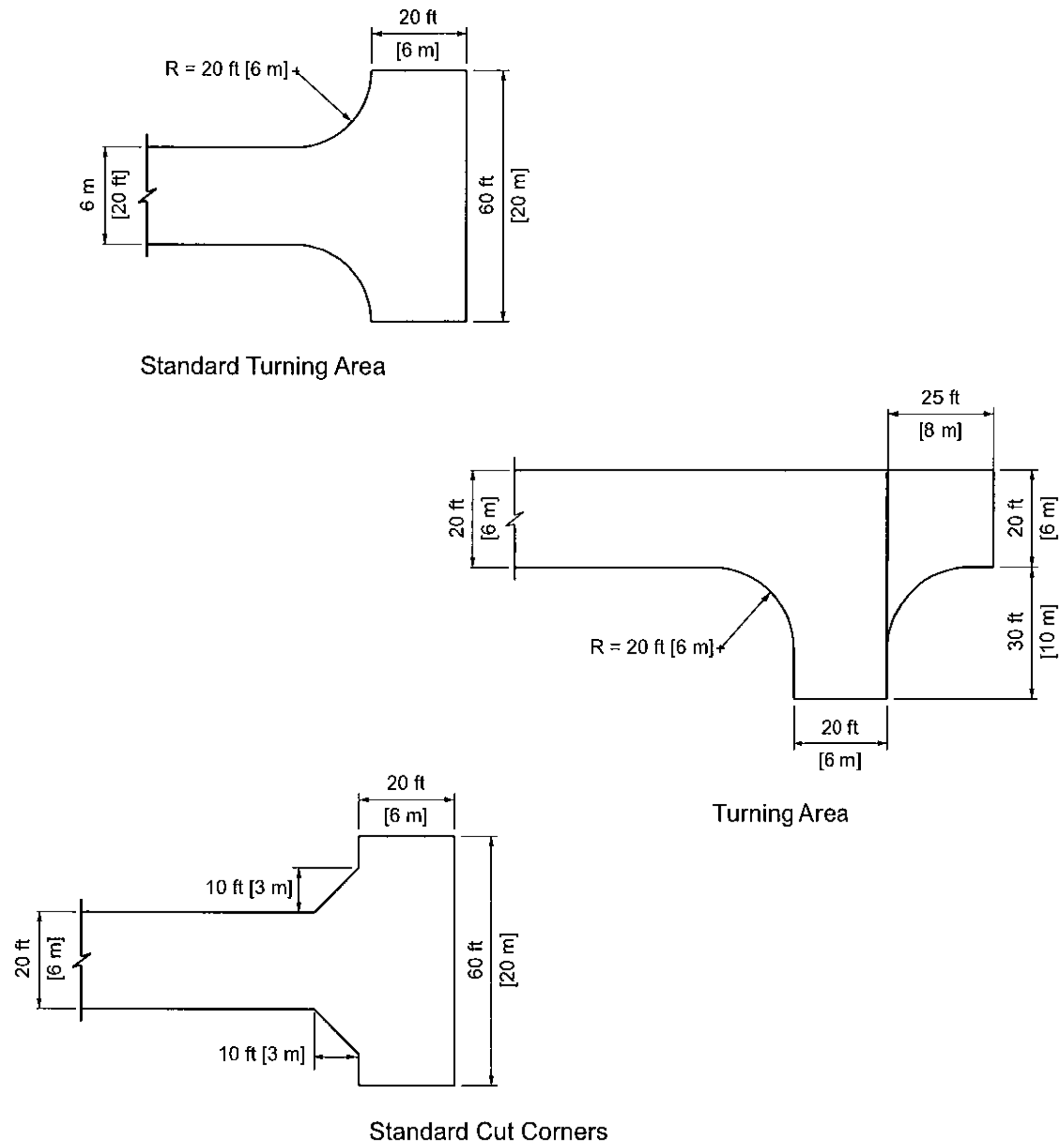


Figure 5-2. Alley Turnarounds

5.3.3 Structures

5.3.3.1 New and Reconstructed Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the current *AASHTO LRFD Bridge Design Specifications* (*9*). The clear width for all new bridges on streets with curbed approaches should be the same as the curb-to-curb width of the approaches. For streets with shoulders and no curbs, the clear roadway width preferably should be the same as the approach roadway width and not less than the width shown in Table 5-6.

Sidewalks on the approaches should be carried across all new structures. There should be at least one sidewalk on all street bridges and desirably on both sides.

5.3.3.2 Vertical Clearance

Vertical clearance at underpasses should be at least 14 ft [4.3 m] over the entire roadway width, with an allowance for future resurfacing. The vertical clearance to sign supports and to bicycle and pedestrian overpasses should be 1.0 ft [0.3 m] greater than the highway structure clearance.

5.3.4 Roadside Design

5.3.4.1 Clear Zones

There is no specific clear zone width applicable to local streets in urban areas. Trees are often located along local streets, preferably on streets with speeds of 40 mph [60 km/h] or less and where adequate sight distance is available at intersecting streets and driveways. Guardrail is not used extensively on local streets except at locations with severe roadside design such as steep foreslopes or approaches to overcrossing structures.

5.3.4.2 Lateral Offset

Lateral offset is defined in Section 4.6.2. Further discussion and suggested guidance on the application of lateral offsets is provided in the AASHTO *Roadside Design Guide* (5).

On all streets a minimum lateral offset of 1.5 ft [0.5 m] should be provided between the curb face and obstructions such as utility poles, lighting poles, and fire hydrants. In areas of dense pedestrian traffic, the construction of vertical curbing (typically 6 to 9 in. [150 to 225 mm] high) aids in delineating areas with high-volume pedestrian traffic.

On facilities without a curb and with a shoulder width less than 4 ft [1.2 m], a minimum lateral offset of 4 ft [1.2 m] from the edge of the traveled way should be provided.

5.3.5 Intersection Design

Intersections, including median openings, should be designed with adequate intersection sight distance, as described in Section 9.5, and the intersection area should be kept free of obstacles. To maintain the minimum sight distance, restrictions on height of embankment, locations of buildings, on-street parking, and screening fences may be appropriate. Any landscaping in the clear-sight triangle should be low growing and should not be higher than 3 ft [1.0 m] above the level of the intersecting street pavements.

Intersecting streets should meet at approximately a 90-degree angle. The alignment design should be adjusted to avoid an angle of intersection of less than 75 degrees. Closely spaced offset intersections should be avoided, whenever practical.

The intersection and approach areas where vehicles are stored while waiting to enter the intersection should be designed with a relatively flat grade; the maximum grade on the approach leg should not exceed 5 percent where practical. Where ice and snow may create poor driving conditions, the desirable grade on the approach leg should be 0.5 percent with no more than 2 percent wherever practical.

At street intersections, there are two distinct radii that need to be considered—the effective turning radius of the turning vehicle and the radius of the curb return (see Figure 5-3). The effective turning radius is the minimum radius appropriate for turning from the right-hand travel lane on the approach street to the appropriate lane of the receiving street. This radius is determined by the selection of a design vehicle appropriate for the streets being designed and the lane on the receiving street into which that design vehicle will turn. Desirably this radius should be at least 25 ft [8 m].

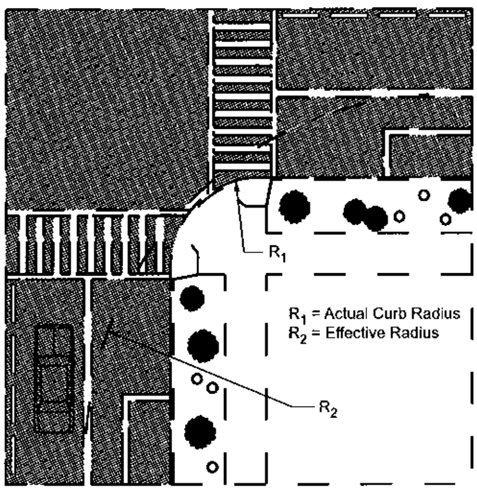


Figure 5-3. Actual Curb Radius and Effective Radius for Right-Turn Movements at Intersections

The radius of the curb return should be no greater than that needed to accommodate the design turning radius. However, the curb return radius should be at least 5 ft [1.5 m] to enable effective use of street-sweeping equipment.

In industrial areas with no on-street parking and few pedestrians, the radius of the curb return should not be less than 30 ft [10 m]; the use of a three-centered curve with sufficiently large radius to accommodate the largest vehicles expected with some frequency is desirable.

Further information pertaining to intersection design appears in Chapter 9.

5.3.6 Railroad–Highway Grade Crossings

Appropriate grade-crossing warning devices should be installed at all railroad–highway grade crossings on local roads and streets. Details of the devices to be used are given in the MUTCD (*12*). In some states, the final approval of the devices to be used may be vested in an agency having oversight over railroads.

Sight distance is an important consideration at railroad–highway grade crossings. There should be sufficient sight distance along the road and railroad tracks for an approaching driver to recognize the crossing, perceive the warning device, determine whether a train is approaching, and stop if necessary. Sufficient sight distance is also needed along the track for drivers of stopped vehicles to decide when it is safe to proceed across the tracks. (For further information on railroad–highway grade crossings, see Section 9.12.) Signalized intersections adjacent to signalized railroad grade crossings should be designed with railroad preemption.

The roadway width at all railroad crossings should be the same as the width of the approach roadway. Sidewalks should be provided at railroad grade crossings to connect existing or future walkways that approach these crossings. Crossings that are located on bicycle routes that are not perpendicular to the railroad may need additional paved shoulder for bicycles to maneuver over the crossing. For further information, see the AASHTO *Guide for the Development of Bicycle Facilities* (*6*).

5.3.7 Traffic Control Devices

Consistent and uniform application of traffic control devices is important. Details of the standard devices and warrants for many conditions are found in the MUTCD (*12*).

Geometric design of streets should fully consider the types of traffic control to be used, especially at intersections where multiphase or actuated traffic signals are likely to be needed.

5.3.8 Roadway Lighting

Drivers need good visibility under day or night conditions to travel along local streets in urban areas. Properly designed and maintained street lighting will produce comfortable and accurate visibility at night, which will facilitate and encourage both vehicular and pedestrian traffic. Thus, where adequate illumination is provided, existing streets can be efficiently used at night. Determinations of need for lighting should be coordinated with crime prevention programs and other community needs.

Warrants for the justification of street lighting involve more than just identifying the functional classification of the roadway. Pedestrian and vehicular volume, night-to-day crash ratios, roadway geometry, merging lanes, curves, and intersections all need careful consideration in establishing illumination levels.

Tables 3.5a (English) and 3.5b (metric) of the AASHTO *Roadway Lighting Design Guide* (*4*) provide recommended minimum levels and uniformity ratios for lighting local roads, alleys, and sidewalks in commercial and residential areas. The ANSI/IESNA RP-8 *American National Standard Practice for Roadway Lighting* (*16*) provides additional discussion on pedestrian and bikeway design criteria, while the FHWA publication entitled *Informational Report on Lighting Design for Midblock Crosswalks* (*15*) provides additional information on nighttime visibility concerns at nonintersection locations.

Because glare also indicates the quality of lighting, the type of fixtures and the height at which the light sources are mounted are also factors in designing street lighting systems. The objectives of the designer should be to minimize visual discomfort and impairment of driver and pedestrian vision due to glare. Where only intersections are lighted, a gradual lighting transition from dark to light to dark should be provided so that drivers may have time to adapt their vision. More detailed discussion of this topic is contained in the *AASHTO Roadway Lighting Design Guide* (*4*) and ANSI/IESNA RP-8 *American National Standard Practice for Roadway Lighting* (*16*).

5.3.9 Drainage

Drainage is an important consideration in urban areas because of high runoff and flood potential. Surface flow from adjacent tributary areas may be intercepted by the street system, where it is collected within the roadway by curbs, gutters, and ditches, and conveyed to an appropriate drainage system. Where drains are available under or near the roadway, the flow is transferred at frequent intervals from the street cross section by gratings or curb-opening inlets to basins and from there by connectors to drainage channels or underground drains.

Economic considerations usually dictate that maximum practical use be made of the street sections for surface drainage. To avoid undesirable flowline conditions, the minimum gutter grade should be 0.30 percent. However, in very flat terrain and where no drainage outlet is available, gutter grades as low as 0.20 percent may be used. Where a drainage system is available, the inlets should be spaced to provide a high level of drainage protection in areas of high pedestrian use or where adjacent property has an unusually important public or community purpose (e.g., schools and churches). For further details, see Section 4.8.2, "Drainage," and see also the *AASHTO Drainage Manual* (*7*).

5.3.10 Erosion Control

Design of streets should consider preservation of natural groundcover and desirable growth of shrubs and trees within the right-of-way. Seeding, mulching, sodding, or other acceptable measures of covering slopes, swales, and other erodible areas should be incorporated in local street design in urban areas. For further information, see Section 3.6.1, "Erosion Control and Landscape Development."

5.3.11 Landscaping

Landscaping in keeping with the character of the street and its environment should be provided for aesthetic and erosion-control purposes. Landscape designs should be arranged to permit a sufficiently wide and clear pedestrian walkway. Individuals with disabilities, bicyclists, and pedestrians should all be considered. Combinations of turf, shrubs, and trees should be considered in continuous border areas along the roadway. However, care should be exercised to observe sight distances and clearance to obstruction guidelines, especially at intersections. The roadside should be developed to serve both the community and the traveling motorist. Landscaping should also consider maintenance problems and costs, future sidewalks, utilities, additional lanes, and possible bicycle facilities.

5.4 RECREATIONAL ROADS

For the purpose of design, highways have been classified in this policy by function with specific design criteria for each functional class. Subsequent chapters discuss the design of collectors, arterials, and freeways. Sections 5.2 and 5.3 discuss the design of typical local roads and streets in rural and urban areas, respectively. Another type of local road, however, is different in purpose and does not fit into any of the classifications identified above. This type of local road is referred to as a special-purpose road and, because of its unique character, separate design criteria are provided. Special-purpose roads include recreational roads, resource recovery roads, and local service roads. Such roads are generally lightly traveled and operate with low traffic speeds and, for these reasons, different design criteria are provided.

5.4.1 General Design Considerations

Roads serving recreational sites and areas are unique by also being part of the recreational experience. Design criteria described in this section meet the unusual demands on roads for access to, through, and within recreational sites, areas, and facilities for the complete enjoyment of the recreationist. The criteria are intended to protect and enhance the existing aesthetic, ecological, environmental, and cultural amenities that form the basis for distinguishing each particular recreational site or area.

Visitors to a recreational site need access to the general area, usually by a statewide or principal arterial highway. Secondly, they need access to the specific recreational site. This is the most important link from the statewide road system. For continuity beyond this point, design criteria assume that the visitor is aware of the recreational nature of the area. The design should be accomplished by a multidisciplinary team of varied backgrounds and experience in order to ultimately provide a road system that is an integral part of the recreational site. Depending on the conditions, internal roadways will have a variety of lower design features.

The criteria discussed in this section are applicable for public roads within all types of recreational sites and areas. Design criteria for recreational roads are discussed for primary access roads, circulation roads, and area roads. Primary access roads are defined as roads that allow through movement into and between access areas. Circulation roads allow movement between activity sites within an access area, whereas area roads allow direct access to individual activity areas, such as campgrounds, park areas, boat launching ramps, picnic groves, and scenic and historic sites.

Figure 5-4 depicts a potential road system serving a recreational area. Road links are labeled in accordance with the classification system noted.

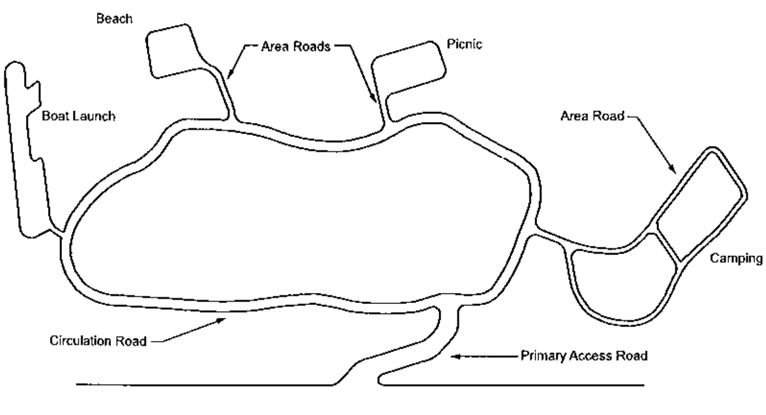


Figure 5-4. Potential Road Network

5.4.1.1 Design Speed

The effect of design speed on various roadway features is considered in its selection; however, the speed is selected primarily on the basis of the character of the terrain and the functional classification of the road. The design speeds should be approximately 40 mph [60 km/h] for primary access roads, 30 mph [50 km/h] for circulation roads, and 20 mph [30 km/h] for area roads. There may be instances where design speeds less than these may be appropriate because of severe terrain conditions or major environmental concerns. Design speeds on one-lane roads are usually less than 30 mph [50 km/h]. If a design speed of greater than 40 mph [60 km/h] is used, Section 5.2, "Local Roads in Rural Areas," should be consulted.

Once a design speed is selected, all geometric features should be related to this speed to obtain a balanced design. Changes in terrain and other physical controls may dictate a change in design speed in certain sections. A decrease in design speed along the road should not be introduced abruptly, but should be extended over a sufficient distance to allow the driver to adjust and make the transition to the slower speed.

5.4.1.2 Design Vehicle

The physical dimensions and operating characteristics of vehicles and the percentage of vehicles of various sizes using recreational roads are primary geometric design controls. Existing and anticipated vehicle types should be reviewed to establish representative vehicles for each functional roadway class. Each design vehicle considered should represent a substantial percentage of the vehicles expected to use the facility during its design life.

Three categories of vehicles are common to recreational areas: motor homes, vehicles with trailers, and standard passenger vehicles. Critical physical dimensions for geometric design are the overall length, width, and height of these units. Minimum turning paths of the design vehicles are influenced by the vehicle steering mechanism, track width, and wheelbase arrangement. Figures in Section 2.8.2 show minimum turn paths for motor homes (MH), passenger cars with 30-ft [9-m] travel trailers (P/T), passenger cars with 20-ft [6.1-m] boats (P/B), and motor homes with 20-ft [6.1-m] boats (MH/B). Turning path dimensions for other vehicle types such as buses and passenger cars are also presented in Section 2.8.2.

5.4.1.3 Grades

Grade design for recreational roads differs substantially from that for other rural highways in that the weight/power ratio of recreational vehicles (RVs) seldom exceeds 50 lb/hp [30 kg/kW]; thus, the grade climbing ability of RVs approaches that for passenger cars. Furthermore, because vehicle operating speeds on recreational roads are relatively low, large speed reductions on grades are not anticipated.

Where grades are kept within the suggested limits, critical length of grade is not a major concern for most recreational roads. Critical length of grade may be a factor on primary access roads into recreational areas, and critical length of grade should be appropriately considered in the design for these roads.

Table 5-7 identifies suggested maximum grades for given terrain and design speed based primarily on the operational performance of vehicles that use recreational roads. Section 3.4.2 contains more detailed information on the selection of an appropriate maximum grade. The erosion resistance of the soil is a major consideration in selection of a maximum grade for a roadway. In many instances, grades considerably less than those shown in Table 5-7 should be chosen to satisfy this concern. In addition, the surface type should also be a factor in grade selection. Steep grades with dirt or gravel surfaces may cause driving problems in the absence of continued maintenance, whereas a paved surface generally will offer better vehicle performance.

Table 5-7. Maximum Grades for Recreational Roads

Type of Terrain	U.S. Customary: Maximum Grade (%) for a Specified Design Speed (mph)						Metric: Maximum Grade (%) for a Specified Design Speed (km/h)				
	15	20	25	30	35	40	20	30	40	50	60
Level	8	8	7	7	7	7	8	8	7	7	7
Rolling	12	11	10	10	9	9	12	11	10	10	9
Mountainous	18	16	15	14	13	12	18	16	15	14	12

5.4.1.4 Vertical Alignment

Vertical curves should be comfortable for the driver, pleasing in appearance, and adequate for drainage. Minimum or greater-than-minimum stopping sight distance should be provided. The designer should consider above-minimum vertical curve lengths at driver decision points, where drainage or aesthetic problems exist, or simply to provide additional sight distance.

Vertical curve design for two-lane roads is discussed in Section 3.4.6, which also presents specific design values. Table 5-8 also includes additional information for very low design speeds not tabulated elsewhere. For two-way, single-lane roads, crest vertical curves should be significantly longer than those for two-lane roads. As previously discussed, the stopping sight distance for a two-way, single-lane road should be approximately twice the stopping sight distance for a comparable two-lane road. Table 5-8 includes *K* values for single-lane roads, from which vertical curve lengths can be determined.

Table 5-8. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves—Recreational Roads

U.S. Customary			
Initial Speed (mph)	Design Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a (ft/%)	
		Crest	Sag
Two-lane roads and one-way, single-lane roads			
15	80	3	10
20	115	7	17
25	155	12	26
30	200	19	37
35	250	29	49
40	305	44	64
Two-way, single-lane roads			
15	160	12	27
20	230	25	44
25	310	45	65
30	400	74	89
35	500	116	117
40	610	172	147

Metric			
Initial Speed (km/h)	Design Stopping Sight Distance (m)	Rate of Vertical Curvature, K^a (m/%)	
		Crest	Sag
Two-lane roads and one-way, single-lane roads			
20	20	1	3
30	35	2	6
40	50	4	9
50	65	7	13
60	85	11	18
Two-way, single-lane roads			
20	40	2	6
30	70	7	13
40	100	15	21
50	130	26	29
60	170	44	40

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Sections 3.2.2 and 3.4.6 for details.)

5.4.1.5 Horizontal Alignment and Superelevation

Because the use of straight sections of roadway would be physically impractical and aesthetically undesirable for many roadways, horizontal curves are essential elements in the design of recreational roads. The proper relationship between design speed and horizontal curvature and the relationship of both to superelevation are discussed in detail in Section 3.3. The guidance provided in Section 3.3 is generally applicable to paved recreational roads; however, in certain instances variations are appropriate. At locations where there is a tendency to drive slowly, as with local and some circulation roads, a maximum superelevation rate of 6 percent is suggested. On roads with design speeds of 20 mph [30 km/h] or less, superelevation may not be warranted.

The design values for maximum curvature and superelevation discussed in Section 3.3 are based on friction data for paved surfaces. Some lower volume recreational facilities may not be paved, and because friction values for gravel surfaces are less than those for paved surfaces, friction values should be considered in curvature selection. Table 5-9 shows appropriate minimum radii for horizontal curves on gravel-surfaced roads for specific design speeds and traction coefficients.

Table 5-9. Guidelines for Minimum Radius of Curvature for New Construction of Unpaved Surfaces with No Superelevation [adapted from (*20*)]

U.S. Customary					
Design speed (mph)	Minimum radius (ft) for specified traction coefficient				
	0.7	0.6	0.5	0.4	0.3
15	50	50	60	75	100
20	75	90	110	135	180
25	120	140	170	210	280
30	170	200	240	300	400
35	235	275	330	410	545
40	305	360	430	535	715
45	390	450	540	675	900

Metric					
Design speed (km/h)	Minimum radius (m) for specified traction coefficient				
	0.7	0.6	0.5	0.4	0.3
20	15	15	15	20	25
30	20	25	30	35	50
40	40	45	50	65	85
50	60	70	80	100	135
60	85	95	115	145	190
70	110	130	155	195	260

5.4.1.6 Sight Distance

Minimum stopping sight distance and passing sight distance are a direct function of the design speed. The subject of sight distance for two-lane roads is addressed in Section 3.2; however, sight distance design criteria are not included in Section 3.2 for roads with very low design speeds and for two-way single-lane roads. On two-way single-lane roads, sufficient sight distance should be available wherever two vehicles might approach one another so that one vehicle can reach the turnout or both vehicles can stop before colliding. Stopping sight distance should be measured using an eye height of 3.5 ft [1.08 m] and a height of opposing vehicle of 4.25 ft [1.30 m]. The stopping sight distance for a two-way, single-lane road should be approximately twice the stopping sight distance that would be used in design of a comparable two-lane road. Suggested stopping sight distances for two-way, single-lane roads are given in Table 5-8.

5.4.1.7 Passing Sight Distance

Because of low operating speeds and the nature of travel on recreational roads, frequent passing maneuvers are not anticipated. Nevertheless, minimum passing sight distance should be provided as frequently as practical, particularly on primary access roads where users travel considerable distances to reach activity sites. Suggested minimum passing sight distances for two-lane recreational roads are given in Table 5-10. Passing sight distance is not a factor on single-lane roads. Where a faster vehicle approaches a slower vehicle from behind, it is assumed that, where appropriate, the slower vehicle will pull into a turnout and allow the faster vehicle to pass.

Table 5-10. Design Controls for Passing Sight Distance for Crest Vertical Curves—Recreational Roads

U.S. Customary		
Design Speed (mph)	Design Passing Sight Distance (ft)	Rate of Vertical Curvature, K^a (ft/%)
20	400	57
25	450	72
30	500	89
35	550	108
40	600	129
45	700	175
50	800	229
55	900	289
60	1,000	357
65	1,100	432

Metric		
Design Speed (km/h)	Design Passing Sight Distance (m)	Rate of Vertical Curvature, K^a (m/%)
30	120	17
40	140	23
50	160	30
60	180	38
70	210	51
80	245	69
90	280	91
100	320	119

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Sections 3.2.4 and 3.4.6 for details.)

5.4.1.8 Cross Slope

Cross slope is provided on roadways for adequate drainage. However, excessive surface sloping can cause steering difficulties. Cross slope rates given in Section 5.2, "Local Roads in Rural Areas," are generally applicable to recreational roads.

5.4.2 Cross-Sectional Elements

5.4.2.1 Width of Roadway

A roadway is defined as the portion of the highway for vehicular use, including shoulders. Appropriate roadway width is selected based on consideration of numerous factors, including existing and anticipated vehicular and bicycle traffic, crash history, terrain, and design speed. Table 5-11 gives recommended traveled-way widths and shoulder widths for the various types of roadways. The sum of the traveled-way and shoulder widths given in Table 5-11 constitutes the roadway width.

The low operating speeds and relatively low traffic volume on recreational roads do not warrant wide shoulders. In addition, wide shoulders may be aesthetically objectionable. These considerations are reflected in the shoulder width values given in Table 5-11. Under adverse terrain conditions, intermittent shoulder sections or turnouts may be suitable alternatives to continuous shoulders, particularly on lower functional roadway classes. Where guardrail is used, the graded width of the shoulder should be increased by about 2 ft [0.6 m].

Table 5-11. Widths of Traveled Way and Shoulders—Recreational Roads

Type of Road	U.S. Customary		Metric	
	Traveled-Way Width (ft)[a]	Shoulder Width (ft)	Traveled-Way Width (m)[a]	Shoulder Width (m)
Primary access roads (two lanes)	22–24	2	6.6–7.2	0.6–1.2
Circulation roads (two lanes)	20–22	2	6.0–6.6	0.6–1.2
Area roads (two lanes)	18–20	0–2	5.4–6.0	0.0–0.6
Area roads (one lane)[b]	12	0–1	3.6	0.0–0.3

[a] Widening on the inside of sharp curves should be provided; additional width equal to 400 [35] divided by the curve radius in feet [meters] is recommended.

[b] Roadway widths greater than 14 ft [4.2 m] should not be used because drivers will tend to use the facility as a two-lane road.

5.4.2.2 Number of Lanes

The number of lanes should be sufficient to accommodate the design traffic volume. For low-volume recreational roads, capacity conditions do not normally govern design, and provision of two travel lanes is appropriate. In some cases where traffic volume is fewer than 100 vehicles per day, it may be practical to use a two-way, single-lane roadway. This type of road is often desirable from economic and environmental standpoints. Where single-lane roadways with two-way traffic are used, turnouts for passing should be provided at intervals. Such turnouts should be provided on all sight-restricted curves, located so that the maximum distance between turnouts is no more than 1,000 ft [300 m], and each turnout should be visible from the adjacent turnouts. For roads that serve substantial proportions of over-wide and extra-long vehicles, the turnout design criteria should be adjusted to accommodate these larger vehicles. Figure 5-5 shows a typical design that may be used for turnouts on tangent and curve sections for two-way, single-lane roads.

W_1	L	W_2
12 ft [3.6 m]	100 ft [30 m]	22 ft [6.6 m]
14 ft [4.2 m]	100 ft [30 m]	24 ft [7.2 m]

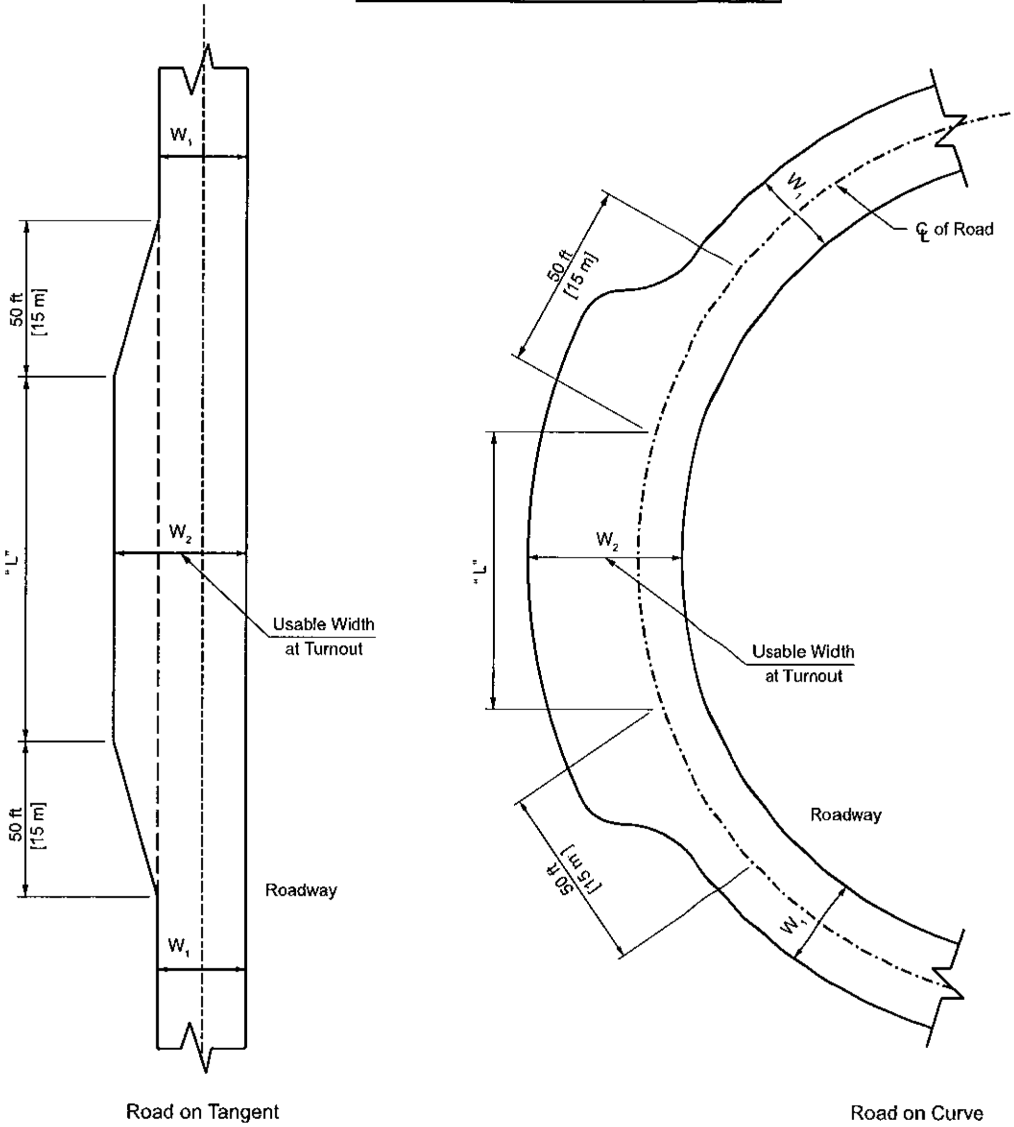


Figure 5-5. Turnout Design

5.4.3 Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the *AASHTO LRFD Bridge Design Specifications* (*9*). The minimum design loading for new bridges should be HL-93 [HL-93]. Higher design loadings are appropriate for highways carrying other than just recreational traffic. The vertical clearance at underpasses should be at least 14 ft [4.3 m] over the entire roadway width. The clear roadway widths for new and reconstructed bridges should be a minimum of the surface width plus 3 ft [1 m]. Where the approach roadway is surfaced for the full crown width, that surfaced width should be carried across structures.

5.4.4 Roadside Design

5.4.4.1 Clear Zones

Providing a clear zone adjacent to a road involves a tradeoff between crash severity potential and aesthetics. A driver who leaves the road should be provided a reasonable opportunity to regain control and avoid serious injury. On the other hand, the philosophy of recreational roads dictates that natural roadside features should be preserved where practical. Because of the character of the traffic and the relatively low operating speeds on recreational roads, wide clear zones are not as important as on high-speed, high-volume facilities. For these reasons, dimensions smaller than those used on these higher order roads are appropriate. Desirably, 10 ft [3 m] or more of recovery area, measured from the edge of the traveled way, should be provided on the higher order recreational roads, (i.e., the primary access roads). These values are recommended for the general case; however, where economic and environmental concerns are great, even smaller values are appropriate. Clear zone widths on the lower order recreational roads, i.e., circulation and area roads, are even less critical than on primary access roads. In areas where the crash potential is greater than normal, such as on the outside of sharp horizontal curves at the end of long, steep downgrades, additional clear zone widths should be provided.

5.4.4.2 Roadside Slopes

Where terrain conditions permit, backslopes, foreslopes, and roadside drainage channels should have gentle well-rounded transitions. Foreslopes of 1V:4H or flatter have lower crash severity potential, are more stable than steeper slopes, and permit establishment and maintenance of turf. The maximum rate of foreslope depends on terrain conditions and the stability of local soils as determined by local experience. Cut sections should be designed with adequate ditches.

The ditch should be deep enough to accommodate the design flow and provide for satisfactory drainage of the pavement base and subbase. While foreslopes of 1V:4H or flatter are preferable, there are other important considerations in ditch design for recreational roads. Surrounding terrain and physical feature preservation may dictate narrow-width ditches. The lower speeds prevailing on recreational roads reduce the chance of personal injury for passengers in vehicles that drive into shallow-sided ditches.

On single-lane roads with low-type surfaces, a crown would not usually be provided. Roads of this type would be inslope graded (toward the cut ditch) or outslope graded (toward the embankment fill), depending on the resistance of the soil to erosion.

5.4.4.3 Roadside Barriers

Roadside barriers should be installed at points of unusual risk, particularly those points that are unusual compared with the overall characteristics of the road. The criteria used in freeway design do not fit the low-volume recreational road situation. The AASHTO *Roadside Design Guide* (*5*) provides some insight into the application of roadside barriers on low-speed, low-volume facilities.

5.4.5 Signing and Marking

The geometric design of a road should be supplemented by standard signing and marking to provide information and warning to drivers. The extent to which signs and markings are used depends on the traffic volume, the type of highway, and the frequency and use by drivers unfamiliar with the area. The MUTCD (*12*) contains details regarding design, location, and application of highway signs and markings.

5.4.6 Bicycle and Pedestrian Facilities

Recreational roads should be reviewed to determine if they are sufficient to accommodate bicycle traffic. Where dedicated bicycle facilities are desired, they should be in accordance with the AASHTO *Guide for the Development of Bicycle Facilities* (*6*).

Where pedestrian facilities are provided, they must be accessible to and usable by individuals with disabilities (*19, 21*); consult the AASHTO *Guide for Planning, Design, and Operation of Pedestrian Facilities* (*2*) and the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*18*) for design elements not addressed in References 19 and 21.

5.5 RESOURCE RECOVERY AND LOCAL SERVICE ROADS

Resource recovery roads include mining and logging roads. Local service roads are those serving isolated areas that have little or no potential for further development (or to need a higher type facility, if developed) and those serving a minimal number of parcels of land. Most of these roads are not through roads (connected to public roads on both ends), but will dead end at the service to the last parcel of land on the road. Design criteria appropriate for these types of roads in many areas are not significantly different from those for recreational roads. For this reason, the criteria developed for recreational roads should be followed to the extent they are applicable. Several items are unique to this category of road and deserve special attention.

Traffic on resource recovery roads is primarily composed of large, slow-moving, heavily loaded vehicles. For this reason, particular attention should be paid to superelevation of horizontal curves. The center of gravity of trucks is much higher than that of passenger cars, and this increases the tendency of trucks to overturn. When semitrailers are used, only part of the payload is on the drive axles. This situation increases the tendency of the drive wheels to spin and sideslip on low-traction surfaces. For these reasons, the maximum superelevation should be limited to 6 percent. On long sustained grades adverse to the direction of haul, the superelevation should be reduced to accommodate slow-moving trucks.

Gradients on this type of facility affect road maintenance costs and costs to users. An economic analysis is usually appropriate to determine the most economical grade for the specific conditions encountered. Such an analysis should consider the increase in culvert installations to prevent ditch erosion on steeper grades and the more frequent surface replacement needs. Adverse grades are a special problem on roads planned for heavy hauling. Sections of adverse grades should not be so long that they slow a loaded truck to crawl speed. Except for short sections that can be overcome largely by momentum, adverse grades merit special analysis. In many instances, failure to use flatter grades may result in additional expenses for transportation during the life of the road that are far in excess of any savings in construction costs.

Geometric design features for resource recovery roads are similar to those for recreational roads in that they should be consistent with the design speed selected. Low design speeds (40 mph [60 km/h] or below) are generally applicable to roads with winding alignment in rolling mountainous terrain. Table 5-12 lists those minimum design speeds for both single-lane and two-lane roads for varying terrain conditions.

Table 5-12. Design Speeds for Resource Recovery and Local Service Roads

Type of Terrain	U.S. Customary: Design Speed (mph) for Roads with Specified Number of Lanes		Metric: Design Speed (km/h) for Roads with Specified Number of Lanes	
	Single Lane	Two Lanes	Single Lane	Two Lanes
Level	30	40	50	60
Rolling	20	30	30	50
Mountainous	10	20	15	30

Because of the mechanical limitations of many of the vehicles using these roads, special attention should be given to the need for warning signs and markings. On long descending grades, consideration should be given to providing escape lanes for use by heavy vehicles that lose their brakes and run out of control. Deceleration may be artificially induced by using loose material or by providing a combination of sufficient length and upgrade for freewheeling deceleration. Further information is provided in Section 3.4.5, "Emergency Escape Ramps."

Many design considerations for resource recovery roads are based on the economics of the equipment operating on the facility. The effects of grades and curvature on operational cost are discussed in considerable detail in the *Logging Road Handbook* (*11*).

In many instances, resource recovery roads are ultimately used for other (e.g., recreational) purposes. In instances such as these, the original design should take into account all the possible ultimate usages.

5.6 LOW-VOLUME ROADS

A low-volume local road is a road that is functionally classified as a local or minor collector road and has a design average daily traffic volume of 2,000 vehicles per day or less. Nearly 80 percent of the roads in the United States can be classified as such. These roads are primarily used by motorists who travel them frequently and are familiar with their geometric design features. The unique characteristics of these roads are generally accepted and anticipated by the drivers using them. Additionally, encounters with others vehicles are infrequent and, statistically, opportunities for multiple-vehicle crashes are unusual. The geometric design of low-volume roads presents a unique challenge because the very low traffic volumes and reduced frequency of crashes make designs normally applied on higher volume roads less cost-effective.

The AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads* (*1*) addresses the unique needs of such roads and the geometric designs appropriate to meet those needs. The AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads* (*1*) may be used in lieu of this publication when designing local roads that fit the applicable criteria. The AASHTO guidelines for low-volume roads address issues for which appropriate geometric design guidance differs from the policies normally applied to higher volume roads. For any geometric design issues not addressed in the AASHTO guidelines for low-volume roads, design professionals should consult Sections 5.2 and 5.3, and Chapter 6.

5.7 REFERENCES

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Chapter 2 – Appendix 2

AASHTO *"A Policy on Geometric Design of Highways and Streets"*

(2018, 7th Edition, 2nd Printing)

Chapter 6: Collector Roads and Streets

6 Collector Roads and Streets

6.1 INTRODUCTION

This chapter presents guidance on the application of geometric design criteria to facilities functionally classified as collector roads and streets. The chapter is subdivided into sections on collectors in rural and urban areas.

A collector is a public road or street, usually serving moderate traffic volumes. There may be few discernible differences between some collectors and local streets, since both collectors and local streets provide access to adjacent residential development and to some neighborhood facilities. However, the design of a collector road or street should reflect its function as a collector and should not be conceived or developed simply to provide continuous access. Collectors should provide access to abutting properties consistent with the level of service desired for all modes of travel.

The function of a collector may be understood by referring to those functional classes that are both higher and lower than the collector classification—the arterial and the local road or street. Since the function of a collector combines aspects of both arterials and local streets, collectors serve a dual function: collecting traffic for movement between arterial streets and local roads, and providing access to abutting properties. Collectors link neighborhoods, areas of homogeneous land use, and mixed use developments with the arterial street system. Collectors not only serve traffic movements between arterials and local streets, but also serve through traffic within local areas. Collectors should be planned so as to not disrupt the activities within the areas they serve.

Every effort should be made to obtain the best practical alignment, profile, sight distance, and drainage that are consistent with terrain, present and anticipated development, project context, current and projected traffic volumes for all transportation modes, crash history, and available funds.

It may not be cost-effective to design minor collector roads and streets that carry 2,000 vehicles per day or fewer using the same criteria applicable to higher volume roads or to make extensive traffic operational or safety improvements to such low-volume roads. Alternate design criteria may be considered for minor collectors that carry 2,000 vehicles per day or fewer in accordance with the AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads*. (*1*).

The specific dimensional design criteria presented in this chapter are appropriate as a guide for new construction of collector roads and streets. Projects to improve existing roads differ from new construction in that the performance of the existing road is known and can guide the design process. Features of the existing design that are performing well may remain in place, while features that are performing poorly should be improved, where practical. Chapter 1 presents a flexible, performance-based design process that can be applied in developing projects on collector roads and streets.

6.2 COLLECTORS IN RURAL AREAS

This section presents guidance on the design of collectors in the rural and rural town contexts. The primary differences between geometric design in the rural and rural town contexts are in the choice of design speed and the increased need in the rural town context to provide parking, to serve increased pedestrian and bicyclist flows, and blend in with the community.

6.2.1 General Design Considerations

Two-lane collectors constitute an important part of the rural highway system. Rural collectors and collectors in rural towns should be designed with the most favorable horizontal alignment, profile, and cross section practical, consistent with traffic volume and topography. Basic information needed for the design of rural collectors includes crash history, both current and projected traffic volumes, terrain, and horizontal and vertical alignment. Design of collectors in rural towns needs additional information such as land use and modal mix that is appropriate to the specific corridor; design of collectors in rural towns is discussed further in Section 6.2.11.

6.2.1.1 Design Speed

Geometric design features should be consistent with a design speed appropriate for the conditions. Low design speeds of 45 mph [70 km/h] and below are generally applicable to collectors in rural towns or collectors with curvilinear alignment in rolling or mountainous terrain, or where environmental conditions make lower speeds appropriate. High design speeds of 50 mph [80 km/h] and above are generally applicable to collectors in the rural context (i.e., outside of rural towns) in level terrain or where environmental conditions are favorable. Table 6-1 identifies minimum design speeds for collector roads in the rural context as a function of the type of terrain and the design traffic volumes. The designer should strive for higher values than those shown where specific crash patterns have been observed that might be reduced and costs are not prohibitive. Refer to Sections 6.2.10 and 6.2.11 for a discussion of speed issues in the rural town context.

Table 6-1. Minimum Design Speeds for Collectors in the Rural Context

Type of Terrain	U.S. Customary			Metric		
	Design speed (mph) for Specified Design Volume (veh/day)			Design speed (km/h) for Specified Design Volume (veh/day)		
	0 to 400	400 to 2,000	over 2,000	0 to 400	400 to 2,000	over 2,000
Level	40	50	60	60	80	100
Rolling	30	40	50	50	60	80
Mountainous	20	30	40	30	50	60

Note: Where practical, design speeds higher than those shown should be considered.

6.2.1.2 Design Traffic Volumes

Rural collectors should be designed to provide acceptable levels of service for current and anticipated future traffic volumes. Usually, the design year is 20 years into the future but may be any number of years within a range from the present (for restoration projects on existing roads) to 20 years (for new or reconstruction projects). The average daily traffic (ADT) volume for the design year should serve as the basis for the project design.

6.2.1.3 Level of Service

In rural areas and in rural towns, level of service C is desirable for collectors. Level of service D is also a practical choice where terrain is rolling or mountainous. For further information, see Section 2.4.5, "Levels of Service," and the *Highway Capacity Manual* (HCM) (*19*).

6.2.1.4 Alignment

The designer should provide the most favorable alignment practical for rural area collectors. Horizontal and vertical alignment should complement each other and should be considered in combination to achieve appropriate safety, capacity, and appearance for the type of improvement proposed. Topography, traffic volume and composition, and right-of-way conditions are controlling features. Abrupt changes in horizontal alignment should be avoided. Vertical curves should meet the sight distance criteria for the design speed. In addition, frequent opportunities for passing should be provided on rural two-lane roads outside of rural towns, where practical. For further information, see Section 3.3, "Horizontal Alignment," and Section 3.4, "Vertical Alignment."

6.2.1.5 Grades

Table 6-2 identifies suggested maximum grades for collectors in rural areas as a function of type of terrain and design speed.

Table 6-2. Maximum Grades for Collectors in Rural Areas

Type of Terrain	U.S. Customary: Maximum Grade (%) for Specified Design Speed (mph)								
	20	25	30	35	40	45	50	55	60
Level	7	7	7	7	7	7	6	6	5
Rolling	10	10	9	9	8	8	7	7	6
Mountainous	12	11	10	10	10	10	9	9	8

Type of Terrain	Metric: Maximum Grade (%) for Specified Design Speed (km/h)							
	30	40	50	60	70	80	90	100
Level	7	7	7	7	7	6	6	5
Rolling	10	10	9	8	8	7	7	6
Mountainous	12	11	10	10	10	9	9	8

Note: Short lengths of grade in rural areas, such as grades less than 500 ft [150 m] in length, one-way downgrades, and grades on low-volume rural collectors (AADT less than 2,000 veh/day) may be up to 2 percent steeper than the grades shown above.

6.2.1.6 Cross Slope

Traveled-way cross slopes provide proper drainage. Normally, cross slopes range from 1.5 to 2 percent for paved roadways. Paved roadways are those that retain smooth riding qualities and good non-skid properties in all weather conditions under heavy traffic volumes and loadings with little maintenance needed.

Unpaved roadways are those with treated earth surfaces and those with loose aggregate surfaces. A cross slope of 3 to 6 percent is desirable for unpaved roadways. For further information, see Section 4.2.2, "Cross Slope."

6.2.1.7 Superelevation

Many rural collectors have curvilinear alignments. A superelevation rate compatible with the design speed should be used. For rural collectors, superelevation should not exceed 12 percent. Where snow and ice conditions may be a factor, the superelevation rate should not exceed 8 percent. Superelevation runoff denotes the length of roadway needed to accomplish a change in outside-lane cross slope from zero (flat) to full superelevation, or vice versa. Adjustments in design runoff lengths may be needed to provide a smooth ride, surface drainage, and good appearance. Section 3.3, "Horizontal Alignment," provides a detailed discussion on superelevation for the various design speeds.

6.2.1.8 Sight Distance

Stopping sight distance and passing sight distance are a direct function of the design speed. An eye height of 3.5 ft [1.08 m] and an object height of 2.0 ft [0.60 m] are used to determine stopping sight distance. An eye height of 3.5 ft [1.08 m] and an object height of 3.5 ft [1.08 m] are used to determine passing sight distance. For further information on sight distance, see Tables 6-3 and 6-4 and Section 3.2, "Sight Distance."

Table 6-3. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves

U.S. Customary			
Design Speed	Design Stopping Sight Distance	Rate of Vertical Curvature, K^a(ft/%)	
(mph)	(ft)	crest	sag
20	115	7	17
25	155	12	26
30	200	19	37
35	250	29	49
40	305	44	64
45	360	61	79
50	425	84	96
55	495	114	115
60	570	151	136
65	645	193	157

Metric			
Design Speed	Design Stopping Sight Distance	Rate of Vertical Curvature, K^a (m/%)	
(km/h)	(m)	Crest	Sag
30	35	2	6
40	50	4	9
50	65	7	13
60	85	11	18
70	105	17	23
80	130	26	30
90	160	39	38
100	185	52	45

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Sections 3.2.2 and 3.4.6 for details.)

Table 6-4. Design Controls for Crest Vertical Curves Based on Passing Sight Distance

U.S. Customary		
Design Speed (mph)	Design Passing Sight Distance (ft)	Rate of Vertical Curvature, K^a (ft/%)
20	400	57
25	450	72
30	500	89
35	550	108
40	600	129
45	700	175
50	800	229
55	900	289
60	1,000	357
65	1,100	432

Metric		
Design Speed (km/h)	Design Passing Sight Distance (m)	Rate of Vertical Curvature, K^a (m/%)
30	120	17
40	140	23
50	160	30
60	180	38
70	210	51
80	245	69
90	280	91
100	320	119

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Sections 3.2.4 and 3.4.6 for details.)

6.2.2 Cross-Sectional Elements

6.2.2.1 Width of Roadway

For paved roadways, the minimum roadway width is the sum of the traveled way and shoulder widths shown in Table 6-5. Graded shoulder width is measured from the edge of the traveled way to the point of intersection of shoulder slope and foreslope. Where roadside barriers are included, a minimum offset of 4 ft [1.2 m] from the traveled way to the barrier should be provided, wherever practical. For further information, see Section 4.4, "Shoulders," Section 4.10.2, "Longitudinal Barriers," and Section 3.3.10, "Traveled-Way Widening on Horizontal Curves" for vehicle offtracking information.

Table 6-5. Minimum Width of Traveled Way and Shoulders

U.S. Customary			
Design Speed (mph)	Minimum Width of Traveled Way (ft) for Specified Design Volume (veh/day)		
	under 400	400 to 2000	over 2000
20	20[a]	20	22
25	20[a]	20	22
30	20[a]	20	22
35	20[a]	22	22
40	20[a]	22	22
45	20	22	22
50	20	22	22
55	22	22	22[b]
60	22	22	22[b]
65	22	22	22[b]
All Speeds	Width of Shoulder on Each Side of Road (ft)		
	2	4	6

Metric			
Design Speed (km/h)	Minimum Width of Traveled Way (m) for Specified Design Volume (veh/day)		
	Under 400	400 to 2000	over 2000
30	6.0[a]	6.0	6.6
40	6.0[a]	6.0	6.6
50	6.0[a]	6.0	6.6
60	6.0[a]	6.6	6.6
70	6.0	6.6	6.6
80	6.0	6.6	6.6
90	6.6	6.6	6.6[b]
100	6.6	6.6	6.6[b]
All Speeds	Width of Shoulder on Each Side of Road (m)		
	0.6	1.5	2.4

[a] An 18-ft [5.4-m] minimum width may be used for roadways with design volumes under 250 veh/day.

[b] Consider using lane width of 24 ft [7.2 m] where substantial truck volumes are present or agricultural equipment frequently uses the road.

Note: See text for roadside barrier and offtracking considerations.

6.2.2.2 Number of Lanes

The number of lanes should be sufficient to accommodate the design traffic volumes for the desired level of service. Normally, capacity conditions do not govern rural collector roads, and two lanes are appropriate. For further information, see Section 2.4, "Highway Capacity."

6.2.2.3 Parking Lanes

Parking lanes are generally not provided on rural collectors, but may be needed on collectors in some rural towns. For additional information on parking lanes, see Section 6.3, "Collectors in Urban Areas."

6.2.2.4 Medians

Medians are generally not provided on rural collectors, but may be appropriate on collectors in some rural towns. For additional information on medians, see Section 6.3, "Collectors in Urban Areas."

6.2.2.5 Right-of-Way Width

Providing right-of-way widths that accommodate construction, adequate drainage, and proper maintenance of a collector road is an important part of the overall design. Wide rights-of-way permit the construction of gentle slopes, resulting in a reduced crash severity potential and accommodating easier and more economical maintenance. The acquisition of sufficient right-of-way at the time of initial construction permits subsequent widening of the roadway and the widening and strengthening of the pavement at a reasonable cost as traffic volumes increase.

In developed areas, it may be necessary to limit the right-of-way width. However, the right-of-way width should not be less than that needed to accommodate all elements of the design cross section, utilities, and appropriate border areas.

6.2.2.6 Bicycle/Pedestrian Facilities

Where bicycle and pedestrian facilities are included as part of the design, refer to the AASHTO *Guide for the Development of Bicycle Facilities* (*6*) and the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*2*).

Curbs and sidewalks are generally not constructed on rural collectors, but may be needed on some collectors in rural towns. See Section 6.3, "Collectors in Urban Areas," for additional information.

6.2.3 Structures

6.2.3.1 New and Reconstructed Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the current *AASHTO LRFD Bridge Design Specifications* (*9*). Except as otherwise indicated in this policy, the dimensional design of structures should be in accordance with these design specifications.

The minimum design loading for bridges on collector roads should be the HL-93 design vehicle live loads. The minimum roadway widths for new and reconstructed bridges should be as shown in Table 6-6.

6.2.3.2 Vertical Clearance

Vertical clearance at underpasses should be at least 14 ft [4.3 m] over the entire roadway width, with an additional allowance for future resurfacing. The vertical clearance to sign supports and to bicycle and pedestrian overpasses should be 1.0 ft [0.3 m] greater than the highway structure clearance.

Table 6-6. Minimum Roadway Widths and Design Loadings for New and Reconstructed Bridges

U.S. Customary			Metric		
Design Volume (veh/day)	Minimum Clear Roadway Width for Bridges[a]	Design Loading Structural Capacity	Design Volume (veh/day)	Minimum Clear Roadway Width for Bridges[a]	Design Loading Structural Capacity
under 400	Traveled way + 2 ft (each side)	HL-93	under 400	Traveled way + 0.6 m (each side)	HL-93
400 to 2,000	Traveled way + 4 ft (each side)[b]	HL-93	400 to 2,000	Traveled way + 1.2 m (each side)[b]	HL-93
over 2,000	Approach road-way (width)[b]	HL-93	over 2,000	Approach road-way (width)[b]	HL-93

a Where the approach roadway width (traveled way plus shoulders) is surfaced, that surface width should be carried across the structures.

b For bridges in excess of 100 ft [30 m] in length, the minimum width of traveled way plus 3 ft [1 m] on each side is acceptable.

6.2.4 Roadside Design

There are two primary considerations for roadside design along the traveled way for rural collectors: clear zones and lateral offset.

6.2.4.1 Clear Zones

In rural environments, where speeds are higher and there are fewer constraints than in urban environments, a clear zone appropriate for the traffic volumes, design speed, and facility type should be provided in accordance with the AASHTO *Roadside Design Guide* (*5*). For low-speed rural collectors, a clear-zone width of 7 to 10 ft [2 to 3 m] is desirable.

6.2.4.2 Lateral Offset

Lateral offset is defined in Section 4.6.2. Further discussion and suggested guidance on the application of lateral offsets is provided in AASHTO *Roadside Design Guide* (*5*).

The full approach width (traveled way plus shoulders) should be carried along the roadway and across bridges and overpasses where practical. To the extent practical, where another highway or railroad passes over the roadway, the overpass should be designed so that the pier or abutment supports, including barrier protection systems, have a lateral offset equal to or greater than the lateral offset on the approach roadway.

On facilities without a curb and with shoulder widths less than 4 ft [1.2 m], a minimum lateral offset of 4 ft [1.2 m] from the edge of the traveled way is desirable and a lateral offset of 1.5 ft [0.5 m] should be provided, where practical.

6.2.4.3 Foreslopes

Roadside slopes should be as flat as practical, taking into consideration other design constraints. Flat foreslopes reduce potential crash severities by providing maneuvering area in emergencies and being more stable than steeper slopes. Flat foreslopes also aid in the establishment of plant growth and simplify maintenance operations. The maximum foreslope rate depends on the stability of local soils as determined by a soils investigation and local experience. Steeper slopes, in combination with roadside barriers, may be used when topography and right-of-way are restrictive and a need is justified.

Drivers who inadvertently leave the traveled way can often recover control of their vehicles if foreslopes are 1V:4H or flatter and shoulders and ditches are well rounded or otherwise made traversable. Such recoverable slopes should be provided where terrain and right-of-way conditions allow.

Where provision of recoverable slopes is not practical, the combinations of rate and height of slope should reduce the crash severity for an out-of-control vehicle. Where high fills, right-of-way restrictions, watercourses, or other problems render such designs impractical, roadside barriers should be considered, in which case the maximum rate of fill slope may be used. Reference should be made to the current edition of the AASHTO *Roadside Design Guide* (5). For further information, see Section 4.10, "Traffic Barriers."

Cut sections should be designed with adequate ditches. Preferably, the foreslope should not be steeper than 1V:3H and, where practical, should be 1V:4H or flatter. The ditch bottom and slopes should be well-rounded, and the backslope should not exceed the maximum rate needed for stability.

6.2.5 Intersection Design

Intersections should be located to avoid steep profile grades and to provide adequate approach sight distance. An intersection should not be situated near a sharp crest vertical curve or on a sharp horizontal curve. Where there is no practical alternative to such a location, the approach sight distance on each leg should be checked and, where practical, backslopes should be flattened and horizontal and vertical curves lengthened, to provide additional sight distance. The driver of a vehicle approaching an intersection should have an unobstructed view of the entire intersection and sufficient lengths of the intersecting roadway to anticipate and avoid potential collisions. Sight distances at intersections with six different types of traffic control are presented in Section 9.5, "Intersection Sight Distance."

Intersections should be designed with corner radii adequate for a selected design vehicle, representing a larger vehicle that is expected to use the intersection with some frequency. For information on minimum turning radii, see Section 9.6, "Turning Roadways and Channelization." Where turning volumes are substantial, speed-change lanes and channelization should be considered.

Intersection legs that operate under stop control should intersect at right angles, wherever practical, and should not intersect at an angle less than 75 degrees. For more information on intersection angle, see Section 9.4.2, "Alignment."

A stopping area that is as level as practical should be provided for approaches on which vehicles may be required to stop.

Chapter 9 presents a discussion of the major aspects of intersection design.

6.2.6 Railroad–Highway Grade Crossings

Appropriate grade crossing warning devices should be installed at railroad–highway grade crossings on collector roads and streets. Details of the devices to be used are given in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*10*). In some states, the final approval of these devices may be vested in an agency having oversight over railroads.

Sight distance is an important consideration at railroad–highway grade crossings. There should be sufficient sight distance along the road and along the railroad tracks for an approaching driver to recognize the railroad crossing, perceive the warning device, determine whether a train is approaching, and stop if necessary. If crossing gates are not provided, adequate sight distance along the track is needed for drivers of stopped vehicles to decide when it is safe to proceed across the tracks. For further information on railroad–highway grade crossings, see Section 9.12, "Railroad–Highway Grade Crossings."

The roadway width at railroad crossings should be the same as the width of the approach roadway.

Crossings that are located on bicycle routes that are not perpendicular to the railroad may need additional paved shoulder width for bicycles to maneuver over the crossing. For further information, see the AASHTO *Guide for the Development of Bicycle Facilities* (*6*).

6.2.7 Traffic Control Devices

Traffic control devices should be applied consistently and uniformly. Details of the standard traffic control devices and warrants for various conditions are found in the MUTCD (*10*). Geometric design of rural collectors should fully consider the types of traffic control to be used, especially at intersections where multiphase or actuated traffic signals are likely to be needed. For further information, see Section 3.6.5, "Traffic Control Devices."

6.2.8 Drainage

Drainage, both on the pavement and from the sides and subsurface, is an important design consideration. Inadequate drainage can lead to high maintenance costs and adverse operational conditions. In areas of significant snowfall, roadways should be designed so that there is sufficient storage space outside the traveled way to accommodate plowed snow and proper drainage for melting conditions. Further guidance can be found in the *AASHTO Drainage Manual* (*7*).

6.2.9 Erosion Control and Landscaping

Consideration should be given to the preservation of the natural groundcover and the growth of shrubs and trees within the right-of-way when designing rural collectors. Shrubs, trees, and other vegetation should be considered in assessing the driver's sight line and the clear zone width. Seeding, mulching, sodding, or other acceptable measures for covering slopes, swales, and other erodible areas should also be considered in the rural collector design. For further information, see Section 3.6.1, "Erosion Control and Landscape Development."

6.2.10 Speed Transitions Entering Rural Towns

Rural collectors provide important connections to and through many rural towns. Where a high-speed rural collector leaves the rural context and enters a rural town or other developed area, there will be a high-speed to low-speed transition zone within which drivers should reduce their speed consistent with the rural town environment. The transition area should be effectively designed to encourage speed reduction because, if drivers do not appropriately reduce speeds, they may create conflicts with other vehicles, pedestrians, and bicyclists and may adversely affect community livability. Design treatments that may be implemented, where appropriate, so that high-speed to low-speed transition zones function more effectively include:

- center islands,
- raised medians,
- roundabouts,
- roadway narrowing,
- lane reductions,

- transverse pavement markings,
- colored pavements, and
- layered landscaping.

The treatments, alone or in combination, encourage drivers to reduce speeds by introducing a changed driving environment in which lower speeds appear appropriate to the driver. Additional details concerning design of transition zones can be found in Section 7.2.19 and in NCHRP Report 737, *Design Guidance for High-Speed to Low-Speed Transition Zones for Rural Highways* (*17*).

6.2.11 Design of Collectors in the Rural Town Context

As noted in Section 6.2.1, design speeds of 45 mph [70 km/h] and below are generally appropriate for collectors in the rural town context. Design speeds and posted speed limits may be decreased in stages as drivers leave the rural environment and approach the center of a rural town. On-street parking is seldom needed on collectors in the rural context, but may be vital to the economic success of businesses in the central portion of a rural town. On-street parking may also help in creating an appropriate low-speed environment within the rural town. Pedestrian and bicyclist flows may increase within rural towns creating a need for pedestrian and bicycle facilities. Rural towns may differ in their appropriate speed environment and needs for parking, pedestrian, and bicycle facilities, just as the suburban, urban, and urban core contexts in urban areas differ. Flexibility in the development of design features is appropriate to meet these varying needs in rural towns. Alternative design approaches and further guidance may be found in the discussion of collectors in urban areas in Section 6.3 and in two relevant publications that address the rural town context: *When Main Street is a State Highway* (*13*) developed by the Maryland Department of Transportation and *Main Street… When a Highway Runs Through It* (*14*), developed by the Oregon Department of Transportation. These two publications are primarily applicable to arterials, but present many design concepts that can also be applied to collectors.

6.3 COLLECTORS IN URBAN AREAS

This section presents guidance on the design of collector streets in urban areas. Collectors in urban areas are designed with a flexible approach to meet the needs of the suburban, urban, and urban core contexts. As a collector street moves from the suburban context to the urban context, and then to the urban core context, the emphasis on maintaining higher vehicle operating speeds decreases, the importance of providing on-street parking in appropriate locations increases, and the pedestrian, bicycle, and transit flows that need to be served, will likely increase. A flexible and balanced design approach to serve all transportation modes appropriately should be applied. The balance among transportation modes may differ between projects based on the demand flows for each transportation mode and established area-wide and corridor plans. The design guidance given below should be adapted to the context and needs of each individual facility and corridor.

6.3.1 General Design Considerations

A collector street is a public facility for vehicular travel and includes the entire area within the right-of-way. Collector streets in the suburban, urban, and urban core contexts also serve bicycle and pedestrian traffic and often accommodate transit and public utility facilities within the right-of-way. The development or improvement of streets should be based on a functional street classification established as part of a comprehensive community development plan. The design criteria should be those for the ultimate planned development.

The function of collectors in suburban, urban, and urban core areas is equally divided between mobility and access. Few cities have effective access control restrictions along collector streets; almost all such streets permit access to abutting properties, except where access rights have been acquired. Many new collectors are planned and constructed with little or no access restriction. However, uncontrolled access may eventually result in the obsolescence of a collector facility. Therefore, it is desirable to manage driveway access to collector streets.

When a major objective of the design is to expedite traffic mobility, there are many additional criteria for which guidelines are appropriate. Such criteria include:

- minimizing conflict points,
- providing adequate storage length for all turning movements,
- minimizing conflicts with pedestrians and bicyclists,
- coordinating driveway locations on opposite sides of the roadway,
- locating signals to meet progression needs, and
- maintaining efficient circulation while providing adequate ingress and egress capacity.

Access control on collector streets should be used so that access points conform to the adopted criteria that are related to safety, location, design, construction, and maintenance. Further guidance on access control will be found in the TRB *Access Management Manual* (*18*).

6.3.1.1 Design Speed

Design speed is a factor in the design of collector streets. For consistency in design, the design speed for suburban collector streets should generally be in the range from 35 to 50 mph [60 to 90 km/h], the design speed for urban collector streets should be in the range from 30 to 40 mph [50 to 60 km/h], and the design speed for urban core collector streets should be in the range from 25 to 35 mph [40 to 60 km/h], depending on available right-of-way, terrain, adjacent development, likely pedestrian presence, and other site controls. See Section, 2.3.6, "Speed"for additional information. Appropriate uses and types of curbs vary with design speed; for further information, see Section 6.3.2.5, "Curbs."

In the typical urban area street grid, closely spaced intersections often limit vehicular speeds and thus make the consideration of design speed of less significance.

6.3.1.2 Design Traffic Volumes

Traffic volumes are a factor in determining the geometric criteria to be used in designing collector streets. It usually is difficult and costly to modify the geometric design of an existing collector street unless provisions are made at the time of initial construction. The design traffic volume should be estimated for at least 10 and preferably 20 years into the future.

6.3.1.3 Level of Service

The choice of the design level and quality of service for a facility involves striking an appropriate balance between the needs of and service levels for motor vehicles, pedestrians, transit, and bicycles; the context, the community; and the degree of confidence in future land use development and trip generation projections. In heavily developed sections of metropolitan areas, the use of Level of Service D may be appropriate, although it may be impractical to achieve even this level of service in constrained settings. While motor-vehicle level of service is calculated in a quantitative manner using numerical formulas, quality of service for pedestrians and bicycles is often a more qualitative analysis and may be a more appropriate process for analyzing facility performance, including accessibility, potential conflicts with motor vehicles, stress, and overall acceptable accommodation. For additional guidance on determining the level of service for all modes for a specific facility, refer to Sections 2.3 and 2.4, the *Highway Capacity Manual* (*19*), and the FHWA *Guidebook for Developing Pedestrian and Bicycle Performance Measures* (*15*).

6.3.1.4 Alignment

Alignment in residential areas should closely fit the existing topography to minimize the need for cuts or fills to achieve appropriate safety, capacity, and appearance.

6.3.1.5 Grades

Grades for collector streets should be as level as practical, consistent with the surrounding terrain.

A minimum grade of 0.3 percent is acceptable to facilitate drainage. However, it is recommended that a grade of 0.5 percent or more be used, where practical, for drainage purposes. Where sidewalks are present, a maximum roadway grade of 5 percent is recommended. Refer to the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*20*) for additional information. The grade of an urban street is generally depressed below the surrounding terrain to direct drainage from adjacent property to the curb area so that it can reach the storm drain system. Applicable gradients, vertical curve lengths, and other pertinent features are addressed in Section 3.4, "Vertical Alignment." Maximum grades for collector streets in the urban and urban core contexts should be as shown in Table 6-7. Maximum grades for higher speed suburban collector streets should be as shown in Table 6-2.

Table 6-7. Maximum Grades for Collector Streets in the Urban and Urban Core Contexts

Type of Terrain	U.S. Customary: Maximum Grade (%) for Specified Design Speed (mph)								
	20	25	30	35	40	45	50	55	60
Level	9	9	9	9	9	8	7	7	6
Rolling	12	12	11	10	10	9	8	8	7
Mountainous	14	13	12	12	12	11	10	10	9

Type of Terrain	Metric: Maximum Grade (%) for Specified Design Speed (km/h)							
	30	40	50	60	70	80	90	100
Level	9	9	9	9	8	7	7	6
Rolling	12	12	11	10	9	8	8	7
Mountainous	14	13	12	12	11	10	10	9

6.3.1.6 Cross Slope

Traveled-way cross slope should be adequate to provide proper drainage. Cross slope should normally be from 1.5 to 3 percent where there are flush shoulders adjacent to the traveled way or where there are outer curbs. For more information on traveled-way and shoulder cross slope, see Sections 4.2.2 and 4.4.3.

6.3.1.7 Superelevation

Superelevation, in specific locations, may be advantageous for collector street traffic operation. However, superelevation may be impractical or undesirable in built-up areas because of the combination of wide pavement areas, proximity of adjacent development, control of cross slope, profile for drainage, frequency of cross streets, and other urban features. Where used, superelevation on collector streets should be 6 percent or less. On suburban collector streets, superelevation should be 12 percent or less and should not exceed 8 percent where snow and ice conditions are a factor. The absence of superelevation on urban collectors for low speeds of 45 mph [70 km/h] and below is generally not detrimental to the motorist. Often, some warping or partial removal or reversal of the pavement crown may facilitate operations. When warping or removing the pavement crown, drainage should be considered. For further information, see Section 3.3, "Horizontal Alignment," including the specific guidance in Section 3.3.6, "Design for Low-Speed Streets in Urban Areas."

6.3.1.8 Sight Distance

Stopping sight distance for collector streets varies with design speed. Design for passing sight distance is seldom appropriate on collector streets. For further information, see Tables 6-3 and 6-4, as well as Section 3.2, "Sight Distance."

6.3.2 Cross-Sectional Elements

6.3.2.1 Width of Roadway

The width of a collector street should be planned as the sum of the widths of the ultimate number of lanes for moving traffic, parking, and bicycles, including median width where appropriate.

Lanes within the traveled way should range in width from 10 to 12 ft [3.0 to 3.6 m]. In industrial areas, lanes may be 12 ft [3.6 m] wide except where lack of space for right-of-way imposes severe limitations; in such cases, lane widths of 11 ft [3.3 m] may be used. Added turning lanes at intersections, where used, should range in width from 10 to 12 ft [3.0 to 3.6 m], depending on the volume of trucks. Where shoulders are provided, roadway widths in accordance with Table 6-5 should be considered. Additional guidance on the width of roadways used by transit vehicles can be found in the AASHTO *Guide for Geometric Design of Transit Facilities on Highways and Streets* (*8*).

6.3.2.2 Number of Lanes

Two traffic lanes are sufficient for most collector streets. In some instances, in commercial areas where there are intersection and midblock left turns, it may be advantageous to provide additional left-turn lanes or a continuous two-way left-turn lane in the center of the roadway. Bicycle lanes are often provided on collector streets to create continuous bicycle networks in the community.

The number of lanes to be provided on collector streets with high traffic volumes should be determined from a capacity analysis. This analysis should consider anticipated transportation modes, and both intersections and midblock locations in assessing the ability of a proposed design to provide the desired level of service for all users. Such analyses should be made for the future design year traffic volume by using the procedures in the most recent edition of the *Highway Capacity Manual* (*19*) or other appropriate traffic analysis tools. For further information, see Section 2.4, "Highway Capacity," and the FHWA *Traffic Analysis Tools* website (*11*),

6.3.2.3 Parking Lanes

Although on-street parking may impede traffic flow and parked vehicles may at times be involved in crashes, provision of parking lanes parallel or angled to the curb is needed to serve adjacent development on many collector streets. Where on-street parking spaces are designated, a portion of spaces should be accessible to persons with disabilities. For more details refer to the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*20*).

Parallel parking is normally acceptable on urban area collectors where sufficient street width is available to provide a parking lane. In residential areas, a parallel parking lane from 7 to 8 ft [2.1 to 2.4 m] in width should be provided on one or both sides of the street, as appropriate for the lot size and density of development. In commercial and industrial areas, parking lane widths should range from 8 to 10 ft [2.4 to 3.0 m] and are usually provided on both sides of the street.

The principal disadvantage of conventional head-in or angle parking, in comparison to back-in head-out diagonal or parallel parking, is the reduced visibility for the driver during the back-out maneuver. Back-in parking also allows for the loading of passengers and cargo to be performed from the sidewalk, rather than near the traveled way. Collector street designs with diagonal or angle parking should only be considered in special cases.

6.3.2.4 Medians

Collector streets designed for four or more lanes should include width for an appropriate median treatment, where practical. For general types of median treatments for collector streets, the following widths may be considered:

- paint-striped separation, 2 to 4 ft [0.6 to 1.2 m] wide;
- narrow raised-curbed sections, 2 to 6 ft [0.6 to 1.8 m] wide;
- raised curbed sections, 10 to 16 ft [3.0 to 4.8 m] wide, providing space for left-turn lanes;
- paint-striped sections, 10 to 16 ft [3.0 to 4.8 m] wide, providing space for two-way left-turn lanes; and
- raised-curb sections, 18 to 25 ft [5.4 to 7.6 m] wide, to provide more space for left-turn lanes and for passenger cars to stop in median openings.

Wider medians from 27 to 40 ft [8 to 12 m] may be used for a parkway design where space is available for landscaping. Each increment in additional median width provides specific operational advantages. Medians should be as wide as practical within the constraints of site conditions.

On collector streets with raised-curb medians, openings should be provided only at intersections with other streets and at reasonably spaced driveways serving major traffic generators such as industrial plants and shopping centers. Median openings should be designed to include left-turn lanes.

The design of collector streets with raised-curb medians should include drainage systems with drainage inlets and catch basins.

Median openings should be located only where there is adequate sight distance. The shape and length of the median openings will vary depending on the width of the median and the vehicle types that are to be accommodated. The minimum length of median openings should be that of the projected roadway width of the intersecting cross street or driveway. Desirably, the length of median openings should be wide enough to provide for the turning radius for the design vehicle for left-turn maneuvers between the inner edge of the lane adjacent to the median and the centerline of the intersecting roadway.

On many collector streets, it may be impractical to use a raised-curb median. A continuous center two-way left-turn lane, flush with the adjacent traveled way, is an alternative design that may be considered. Where pedestrian crossings are anticipated, intermittent median islands that provide pedestrian refuge are recommended. A further discussion on medians is found in Section 4.11, "Medians" and in Section 9.8, "Median Openings."

6.3.2.5 Curbs

Collector streets are normally designed with curbs to allow greater use of available width and for control of drainage, protection of pedestrians, and delineation. The curb on the side of the traveled way may be a vertical curb, 6 in. [150 mm] high, usually with an appropriate batter for low-speed roadways. A vertical curb should not be used on roadways with speeds greater than 45 mph [70 km/h]; sloping curbs with heights of 6 in. [150 mm] or less may be used in this situation. A sloping curb with a height of 4 in. [100 mm] should be considered on higher speed facilities with infrequent accesses and intersecting streets.

On divided streets, the type of median curbs should be determined in conjunction with the median width and the type of turning movement control to be provided. Where midblock left-turn movements are permitted and the median width is less than 10 ft [3 m], a well-delineated flush or rounded raised median separator 2 to 4 in. [50 to 100 mm] high is effective in channelizing traffic and in avoiding excessive travel distances and concentrations of turns at intersections. Where wider traversable medians are appropriate, they may be either flush or bordered with low curbs 1 to 2 in. [5 to 50 mm] high. On narrow and intermediate-width medians, and on some wide medians, where cross-median movements are undesirable, a curb should be used on the median side of the traveled way. Consideration of the type (vertical or sloped) and height should be based on the roadway speed and other factors as stated above. For further information, see Section 4.7, "Curbs," and the AASHTO *Roadside Design Guide* (*5*).

Vertical curbs with heights of 6 in. [150 mm] or more, adjacent to the traveled way, should be offset a minimum of 1 to 2 ft [0.3 to 0.6 m] from the edge of the traveled way. Where there is combination curb-and-gutter construction, the gutter pan width, which is normally 1 to 2 ft [0.3 to 0.6 m], may provide the offset distance.

Where pedestrian crosswalks are provided for crossing a collector street, they must be accessible through any medians that are present. See Section 4.17.3 for further discussion.

6.3.2.6 Right-of-Way Width

The right-of-way width for collector streets should be sufficient to accommodate the ultimate planned roadway, including the median, parking lanes, shoulders, border areas, sidewalks, bicycle facilities, public utilities, and outer slopes. The width of right-of-way for two-lane collector streets should generally range from 40 to 80 ft [12 to 25 m], depending on these items.

6.3.2.7 Provision for Utilities

In addition to the primary purpose of serving vehicular traffic, collector streets may accommodate public utility facilities within the street right-of-way in accordance with state law or municipal ordinance. Use of the right-of-way by utilities should be planned to minimize interference with traffic using the street. The AASHTO *Guide for Accommodating Utilities within Highway Right-of-Way* (*3*) presents general principles for utility location and construction to minimize conflicts between the use of the right-of-way for vehicular movements and the secondary objective of providing space for locating utilities. Utilities must be located such that they do not make pedestrian facilities inaccessible.

6.3.2.8 Border Area

The border area between the roadway and the right-of-way line should be wide enough to serve several purposes, including serving as a buffer space between pedestrians, bicyclists, and vehicular traffic; a sidewalk; and an area for underground and aboveground utilities such as traffic signals, parking meters, and fire hydrants. A portion of the border area should accommodate snow storage and may include aesthetic features such as grass or landscaping. The border width should be at least 12 ft [3.6 m], including the sidewalk width. Traffic signals, utility poles, fire hydrants, and other utilities should be placed as far back from the curb as practical to reduce the likelihood of being struck by vehicles that run off the road. Breakaway features also may be built into such obstacles, where practical, to reduce the severity of collisions that may occur.

6.3.2.9 Bicycle/Pedestrian Facilities

Where bicycle facilities are provided, refer to the AASHTO *Guide for the Development of Bicycle Facilities* (*6*) for design guidance.

Sidewalks should be provided along both sides of collector streets that are used for pedestrian access to schools, parks, shopping areas, and transit stops. Sidewalks are desirable on both sides of collector streets. The sidewalk should be located as far as practical from the traveled way, usually close to the right-of-way line. The minimum sidewalk width should be at least 4 ft [1.2 m] with 5 ft [1.5-m] passing areas every 200 ft [61 m] in residential areas and should range from 4 to 12 ft [1.2 to 7.2 m] in commercial areas. Sidewalk widths of at least 5 ft [1.5 m] are recommended. For further information, see Section 4.17.1, "Sidewalks." Where pedestrian facilities are provided, they must be accessible to and usable by individuals with disabilities (*21, 22*). Additional design guidance on sidewalks can also be found in the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*2*) and the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*20*).

Curb ramps must be provided at all marked and unmarked crosswalks to accommodate persons with disabilities. Section 4.17.3, "Curb Ramps," discusses various design applications for such ramps.

6.3.2.10 Driveways

The width of a driveway entrance, placement with respect to property lines and intersecting streets, angle of entrance, vertical alignment, and the number of entrances to a single property should be controlled. Where driveways cross sidewalks, the sidewalk must be accessible to and usable by individuals with disabilities (*21, 22*). Further guidance on the design of sidewalk-driveway interfaces can be found in the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*2*). Additional guidance on design of driveways can be found in Section 4.15.2, "Driveways."

6.3.3 Structures

6.3.3.1 New and Reconstructed Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the current *AASHTO LRFD Bridge Design Specifications* (*9*). The clear width for new bridges on urban collector streets with curbed approaches should be the same as the curb-to-curb width of the approach roadway. The bridge rail should be flush with the front face of the curb if no sidewalk is present to minimize the likelihood that vehicles will vault the rail. For collector streets with shoulders and no curbs, the full width of approach roadways should preferably be extended across bridges. Sidewalks on the approaches should be extended across new structures. Due to the long design life of bridges, sidewalks should be provided on both sides on bridges on collector streets unless a separate pedestrian bridge is provided. Access to the sidewalk must be provided for all pedestrians, including those with disabilities. Further discussion of roadway widths for bridges is presented in Section 4.10, "Traffic Barriers." Table 6-6 applies to bridge widths on urban collector streets.

6.3.3.2 Vertical Clearance

Vertical clearance at underpasses should be at least 14 ft [4.3 m] over the entire roadway width, with an additional allowance for future resurfacing. The vertical clearance to sign supports and to bicycle and pedestrian overpasses should be 1.0 ft [0.3 m] greater than the highway structure clearance.

6.3.4 Roadside Design

There are two primary considerations for roadside design along the traveled way—clear zones and lateral offset.

6.3.4.1 Clear Zones

In an urban environment, right-of-way is often extremely limited and in many cases it is not practical to establish a full-width clear zone. Urban environments are characterized by curbs, sidewalks, enclosed drainage, numerous fixed objects (e.g., signs, utility poles, luminaire supports, fire hydrants, street furniture, etc.) and frequent traffic stops. These environments typically have lower operating speeds and, in many instances, on-street parking is provided. Where establishing a full-width clear zone in an urban area is not practical due to right-of-way constraints, consideration should be given to establishing a reduced clear zone or incorporating as many clear-zone concepts as practical, such as removing roadside objects or making them crashworthy. Refer to the guidance in the AASHTO *Roadside Design Guide* (*5*) for additional discussion on roadside design limitations in urban environments.

6.3.4.2 Lateral Offset

Lateral offset is defined in Section 4.6.2. Further discussion and suggested guidance on the application of lateral offsets is provided in AASHTO *Roadside Design Guide* (*5*).

For collectors in urban environments, a lateral offset is needed to vertical obstructions (signs, utility poles, luminaire supports, fire hydrants, etc., including breakaway devices) to accommodate motorists operating on the highway. This lateral offset to obstructions helps to:

- avoid drivers shying away from obstructions and vehicle encroachments into opposing or adjacent lanes;
- improve driveway and horizontal sight distances;
- reduce the travel lane encroachments from occasional parked and disabled vehicles;
- improve travel lane capacity; and
- minimize contact between obstructions and vehicle mirrors, car doors, and trucks that overhang the edge of the pavement when turning.

Where a curb is present, the lateral offset is measured from the face of curb. A minimum lateral offset of 1.5 ft [0.5 m] should be provided from the face of the curb with 3 ft [1 m] at intersections to accommodate turning trucks and improve sight distance. Consideration should be given to providing more than the minimum lateral offset to obstructions, where practical, by placing fixed objects behind the sidewalk. Traffic barriers should be located in accordance with the AASHTO *Roadside Design Guide* (*5*).

On curbed facilities located in suburban areas, there may be an opportunity to provide greater lateral offset in the location of fixed objects. These facilities are generally characterized by higher operating speeds and have sidewalks separated from the curb by a border area. Although establishing a clear zone commensurate with the suggested values in the AASHTO *Roadside Design Guide* (5) may not be practical due to right-of-way constraints, consideration should be given to establishing a reduced clear zone or incorporating as many clear zone concepts as practical, such as removing roadside objects or making them crashworthy.

On facilities without a curb and with shoulder widths less than 4 ft [1.2 m], a minimum lateral offset of 4 ft [1.2 m] from the edge of the traveled way is desirable.

6.3.5 Intersection Design

The pattern of traffic movements at intersections and the volume of traffic on each approach, including pedestrian and bicycle traffic, are indicative of the appropriate type of traffic control devices, the widths of lanes (including auxiliary lanes), and where applicable, the type and extent of channelization needed to accommodate all anticipated users. Designing for peak flows of motorized travel may compromise the usability of the intersection for other transportation modes throughout the day. The arrangement of islands and the shape and length of auxiliary lanes may differ depending on whether or not signal control is used. The composition and character of traffic is a design control; movements involving large trucks need larger intersection areas and flatter approach grades than those used at intersections where traffic consists predominantly of passenger cars. Bus stops located near an intersection may create a need for additional modification to the intersection design. Traffic approach speed has an effect on the geometric design as well as on the appropriate traffic control devices and pavement markings. For further information, see Section 3.6.5, "Traffic Control Devices."

The number and location of approach roadways and their angles of intersection are major controls for intersection geometric design, the location of islands, and the types of control devices, except where roundabouts are provided. Intersections at grade preferably should be limited to no more than four approach legs. When two crossroads intersect the collector highway in close proximity, they should be combined into a single intersection.

Important design considerations for at-grade intersections fall into two major categories—the geometric design of the intersection (including a capacity analysis) and, except where roundabouts are provided, the location and type of traffic control devices. Generally, these considerations are applicable to both new and existing intersections, although for existing intersections in built-up areas, heavy development may make extensive design changes impractical.

Chapter 9 presents a discussion of the major aspects of intersection design.

6.3.6 Railroad–Highway Grade Crossings

Appropriate grade crossing warning devices should be installed at railroad–highway grade crossings on collector streets. Details of these devices are given in the MUTCD (*10*). In some states, the final approval of these devices may be vested in an agency having oversight over railroads.

Sight distance is an important consideration at railroad–highway grade crossings on collector streets. There should be sufficient sight distance along the street for the approaching driver to recognize the railroad crossing, perceive the warning device, determine whether a train is approaching, and stop if necessary. At railroad–highway grade crossings without gates, adequate sight distance along the tracks is also needed for drivers of stopped vehicles to decide when it is safe to proceed across the tracks.

The roadway width at crossings should be the same as the curb-to-curb width of the approaches. Where street sections are not curbed, the crossing width should be consistent with the approach street and shoulder widths. Sidewalks should continue across railroad grade crossings where approach sidewalks exist or are planned within the near future. Provisions for future sidewalks should be incorporated into design, if they can be anticipated, to avoid future crossing work on the railroad facility.

Crossings that are located on bicycle routes that are not perpendicular to the railroad may need additional paved shoulder width for bicycles to maneuver over the crossing. For further information, see the AASHTO *Guide for the Development of Bicycle Facilities* (*6*).

The design of railroad–highway grade crossings is discussed more fully in Section 9.12.

6.3.7 Traffic Control Devices

Traffic control devices should be applied consistently and uniformly. Details of the standard devices and warrants for various conditions are found in the MUTCD (*10*).

Geometric design of collector streets should fully consider the types of traffic control to be provided, especially at intersections where multiphase or actuated traffic signals are likely to be needed. Signal progression, signal phasing (including pedestrian and bicycle phases), and traffic flow rates are important considerations in signalized intersection design. For further information, see Section 3.6.5, "Traffic Control Devices."

6.3.8 Roadway Lighting

Good visibility under both day and night conditions is fundamental to enable motorists, pedestrians, and bicyclists to travel on roadways in a safe and coordinated manner. Properly designed and maintained street lighting provides comfortable and accurate night visibility, which should facilitate vehicular, bicycle, and pedestrian traffic.

Decisions concerning appropriate street lighting should be coordinated with public safety management, crime prevention, and other community concerns. The AASHTO *Roadway Lighting Design Guide* (*4*) provides discussion on street and roadway lighting. Further information is also provided in Section 3.6.3, "Lighting," the ANSI/IESNA RP-8 *American Standard Practice for Roadway Lighting* (*16*), and the FHWA *Informational Report on Lighting Design for Midblock Crosswalks* (*12*).

6.3.9 Drainage

Surface runoff is gathered by a system of gutters, inlets, catch basins, and storm sewers. The gutter grade should be at least 0.3 percent. However, a gutter grade of 0.5 percent or more should be provided where practical, for better drainage. Inlets or catch basins with an open grate should be located in the gutter line and be spaced so that ponding of water on the pavement does not exceed tolerable limits. In addition, grates should be designed to accommodate bicycle and pedestrian traffic. For additional details, see Section 4.2, "Traveled Way"; Section 4.4, "Shoulders"; Section 4.7, "Curbs"; and Section 4.8, "Drainage Channels and Sideslopes."

6.3.10 Erosion Control

Consideration should be given to preserving the natural groundcover and the growth of shrubs and trees within the right-of-way when designing urban collectors. Seeding, mulching, sodding, or other acceptable measures for covering slopes, swales, and other erodible areas should also be considered in urban collector street design. For further information, see Section 3.6.1, "Erosion Control and Landscape Development."

6.3.11 Landscaping

Landscaping should be provided in keeping with the character of the street and its environment for both aesthetic and erosion control purposes. Landscape designs should be arranged to permit a sufficiently wide, clear, and accessible pedestrian walkway. The needs of individuals with disabilities, bicyclists, and pedestrians should be considered. Combinations of turf, shrubs, and trees should be considered in continuous border areas along the roadway. However, care should be exercised to provide sight distances, lateral offset, and clear zones, especially at intersections. The roadside should be developed to serve both the community and the motorist. Landscaping should also consider maintenance operations and costs, future sidewalks, utilities, additional lanes, and bicycle facilities.

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Chapter 2 – Appendix 3

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Chapter 7: Rural and Urban Arterials

7 Arterial Roads and Streets

7.1 INTRODUCTION

The principal and minor arterial road systems provide for travel between major points in both rural and urban areas. Within urban areas, the arterial road system often operates at lower speeds and plays an important role in inter- and intra-urban circulation networks. Chapter 1 discusses extensively the functional purposes of arterials in both rural and urban areas with the exception of grade-separated freeways and expressways, which are covered in Chapter 8. This chapter provides the general information needed to establish the basis of design for arterials in rural and urban areas.

The design of arterials covers a broad range of roadways, from two-lane to multilane, and is the most difficult class of roadway design because of the need to provide both safe and efficient operations; allow varying degrees of accessibility to adjoining properties; often serve pedestrians, bicyclists, and transit service as well as motor vehicles; and perform effectively under sometimes unusual or constrained conditions. Chapter 1 introduces five contexts for design of roads and streets, to supplement the rural and urban area types that have traditionally been considered. Rural areas consist of two context categories—rural and rural town. Urban areas consist of three context categories—suburban, urban, and urban core.

The designer should be thoroughly familiar with the material in all chapters of this policy in order to skillfully apply design flexibility in blending the various types of arterials into the functional network and surrounding context. Although freeways are included within the functional description of an arterial, they have distinctive design criteria and are therefore treated separately in Chapter 8. For the purposes of this chapter, guidance for arterials in urban areas applies to arterials in the suburban, urban, and urban core contexts.

This chapter considers arterials in rural areas separately from arterials in urban areas because each type of arterial has distinctive features. However, the designer should be prepared to use design features from both arterial types to provide for suitable transitions as an arterial transitions between rural and urban areas, as well as between the varying contexts within those areas.

The specific dimensional design criteria presented in this chapter are appropriate as a guide for new construction of arterial roads and streets. Projects to improve existing roads differ from new construction in that the performance of the existing road is known and can guide the design process. Features of the existing design that are performing well may remain in place, while features that are performing poorly should be improved, where practical. Chapter 1 presents a flexible, performance-based design process that can be applied in developing projects on arterial roads and streets.

7.2 ARTERIALS IN RURAL AREAS

This section presents guidance on the design of arterials in the rural and rural town contexts. The primary differences between geometric design in the rural and rural town contexts are in the choice of design speed and the increased need in the rural town context to provide parking, to serve increased pedestrian and bicyclist flows, and blend in with the community.

7.2.1 General Characteristics

Arterials in rural areas constitute an important part of the rural highway system, including cross sections that range from two-lane roadways to multilane, divided controlled-access highways. The first portion of this chapter relates to the design of arterials in rural areas and the reconstruction of arterials in rural areas. Such roadways are designed on the basis of traffic volume needs and should be constructed to the most favorable design criteria practical.

Principal arterials in rural areas include rural freeways, which are covered in Chapter 8. They also include other multilane roadways and some two-lane highways that connect urban centers and pass through a rural town context. Minor arterials in rural areas link urban centers to larger towns and are spaced to provide a relatively high level of service to developed areas of a state.

For the purposes of this section, an arterial in a rural area that passes through a rural town context will generally have lower speeds along with an increased density of intersections and driveways generating higher levels of vehicle turning movements. There may also be an increased presence of traffic control (such as stop-controlled or signalized intersections), on-street parking, and pedestrian and/or bicycle activity. Design of arterials for the rural town context is addressed in Section 7.2.20.

The appropriate design geometrics for an arterial may be readily determined from the selected design speed and the design traffic volumes of all modes, with consideration of the type of terrain, the general character of the alignment, the composition of traffic and user modes, and the adjacent land use and context. Operational characteristics, design features, cross sections, and rights-of-way are also discussed in this chapter.

Two-lane arterials constitute the majority of the arterial system in rural areas. They generally have all-weather surfaces and are marked and signed in accordance with the current edition of the *Manual on Uniform Traffic Control Devices* (*MUTCD*) (*12*).

7.2.2 General Design Considerations

Basic information needed for the design of rural arterials includes crash history, traffic volumes (both current and projected), terrain, and horizontal and vertical alignment. Design of arterials in rural towns needs additional information such as land use and modal mix that is appropriate to the specific corridor.

7.2.2.1 Design Speed

Design speeds for arterials in rural areas differ between the two rural area contexts—rural context and rural town context. Design speeds for arterials in the rural context are generally greater than 45 mph [70 km/h] and largely depend on terrain, driver expectancy and, in the case of reconstruction projects, the alignment of the existing facility. Design speeds of 50 to 75 mph [80 to 120 km/h] are normally used in level terrain; design speeds of 50 to 65 mph [80 to 100 km/h] are normally used in rolling terrain; and design speeds of 45 to 60 mph [70 to 80 km/h] are used in mountainous terrain. Design speeds for arterials in the rural town context are lower and generally range from 20 to 45 mph [50 to 70 km/h], depending on the activity levels for transportation modes and other goals of the community.

Considerable attention should be given to the transition of high to low speeds on arterials in rural areas as the adjacent land use changes from a rural context to a rural town context. Arterials in the rural context are designed to facilitate high-speed, longer-distance travel. Arterials in the rural town context typically have lower design speeds, increased traffic control, on-street parking, frequent access points, and more pedestrian activity than arterials in the rural context. Drivers need well-designed transition zones that encourage gradual, smooth reductions in speed as they transition from the rural context to the rural town context. Guidance on designing transition zones is presented in Section 7.2.19.

7.2.2.2 Design Traffic Volumes

Before an existing arterial in a rural area is improved or a new arterial in a rural area is constructed, the design traffic volume for motor vehicles should be determined. The first step in determining the design traffic volume is to determine the current average daily traffic (ADT) volume for the roadway; in the case of new construction, the ADT can be estimated. These ADT values should then be projected to the design year, usually 20 years into the future. The design of low-volume arterials in rural areas is typically based on ADT values alone because neither capacity nor intersection operations typically govern the overall operation. Such roadways normally provide free flow under all conditions. By contrast, it is usually appropriate to design high-volume arterials in rural areas using an hourly volume as the design traffic volume. The design hourly volume (DHV) that should generally be used in design is the 30th highest hourly volume of the year, abbreviated as 30 HV, which is typically about 15 percent of the ADT on rural roads. Where an arterial in a rural area has existing pedestrian or bicycle activity or passes through a rural town context, current and projected design volumes should also be estimated for those other roadway users. For further information on the determination of design traffic volumes, see Section 2.3, "Traffic Characteristics."

7.2.2.3 Level of Service

Procedures for estimating the traffic operational performance of particular highway designs are presented in the *Highway Capacity Manual* (HCM) (*35*), which also presents a thorough discussion of the level-of-service concept, including level of service for motor vehicles, pedestrians, and bicyclists. Although the choice of an appropriate design level of service is left to the highway agency, designers should strive to provide the highest level of service practical and consistent with anticipated conditions. Level-of-service characteristics are discussed in Section 2.4.5 and summarized in Table 2-2. For acceptable degrees of congestion, arterials in rural areas and their auxiliary facilities (e.g., turning lanes, passing sections, weaving sections, intersections, and interchanges) should generally be designed for level of service B, except in mountainous areas where level of service C is acceptable. Where arterials in rural areas pass through a rural town context and nonmotorized roadway users are present, or likely to be present in the future, the motor-vehicle level of service may be reduced to provide a more balanced level of service and better accommodate other modes.

7.2.2.4 Sight Distance

Sight distance is directly related to and varies appreciably with design speed. Stopping sight distance should be provided throughout the length of the roadway. Passing and decision sight distances influence roadway operations and should be provided wherever practical. Providing decision sight distance at locations where complex decisions are made greatly enhances the capability for drivers to accomplish maneuvers. Examples of locations where complex decisions are needed include interchanges, high-volume intersections, transitions in roadway width, and transitions in the number of lanes. Providing adequate sight distance on arterials in rural areas, which may combine both high speeds and high traffic volumes, can be complex. Table 7-1 presents the recommended minimum values of stopping and passing sight distance. Refer to Section 3.2 for a comprehensive discussion of sight distance and for tabulated values for decision sight distance.

Table 7-1. Minimum Sight Distances for Arterials in Rural Areas

U.S. Customary		
Design Speed (mph)	Minimum Stopping Sight Distance (ft)	Minimum Passing Sight Distance (ft)
20	115	400
25	155	450
30	200	500
35	250	550
40	305	600
45	360	700
50	425	800
55	495	900
60	570	1000
65	645	1100
70	730	1200
75	820	1300
80	910	1400

Metric		
Design Speed (km/h)	Minimum Stopping Sight Distance (m)	Minimum Passing Sight Distance (m)
30	35	120
40	50	140
50	65	160
60	85	180
70	105	210
80	130	245
90	160	280
100	185	320
110	220	355
120	250	395
130	285	440

Ideally, intersections and railroad crossings should be grade separated or provided with adequate sight distance. Intersections should be placed in sag or tangent locations, where practical, to provide maximum visibility of the roadway, signs, and pavement markings.

7.2.2.5 Alignment

A smooth flowing alignment is desirable on an arterial in a rural area. Changes in alignment, both horizontal and vertical, should be sufficiently gradual to avoid surprising the driver. Minimum radii should be used sparingly; short horizontal curves—particularly at the end of long tangents—should be avoided. Roads with consistent alignment usually function more efficiently and with lower crash rates than roads with poor alignment, even where enhanced signing and pavement marking are provided.

7.2.2.6 Grades

The length and steepness of grades directly affect the operational characteristics of an arterial in a rural area. Table 7-2 presents recommended maximum grades for arterials in rural areas. When vertical curves for stopping sight distance are considered, there are seldom advantages to using the maximum grade values except when grades are long.

Table 7-2. Maximum Grades for Arterials in Rural Areas

Type of Terrain	U.S. Customary: Maximum Grade (%) for Specified Design Speed (mph)									
	20	25	30	35	40	45	50	55	60	65 and above
Level	5	5	5	5	5	5	4	4	3	3
Rolling	8	8	7	7	6	6	5	5	4	4
Mountainous	10	9	8	8	8	7	7	6	6	5

Type of Terrain	Metric: Maximum Grade (%) for Specified Design Speed (km/h)								
	30	40	50	60	70	80	90	100	110 and above
Level	5	5	5	5	5	4	4	3	3
Rolling	8	8	7	6	6	5	5	4	4
Mountainous	10	9	8	8	7	7	6	6	5

7.2.2.7 Cross Slope

Cross slope is provided to enhance roadway drainage. Two-lane rural roadways are normally designed with a centerline crown and traveled-way cross slopes ranging from 1.5 to 2 percent with the higher values being most prevalent.

7.2.2.8 Superelevation

Where curves are used on an arterial in a rural area, a superelevation rate based on the design speed should be used. Superelevation rates should not exceed 12 percent; however, where ice and snow conditions are a factor, the maximum superelevation rate should not exceed 8 percent. The maximum cross-slope break between the traveled way and the shoulder should be limited to 8 percent to reduce the risk of vehicle rollover (*32*). Superelevation runoff consists of the length of roadway needed to accomplish the change in outside-lane cross slope from zero (flat) to a fully superelevated section, or vice versa. Adjustments in design runoff lengths may be needed for smooth riding, drainage, and appearance. Section 3.3 provides a detailed discussion of superelevation and tables of appropriate superelevation rates and runoff lengths for various design speeds.

7.2.3 Cross-Sectional Elements

7.2.3.1 Roadway Width

The logical approach to determining appropriate lane and shoulder widths is to provide a width related to the traffic demands. Table 7-3 provides values for the width of traveled way and usable shoulder that should be considered for the motor-vehicle volumes and design speeds indicated. In addition, the types of vehicles being served (such as freight and bicycles), availability of right-of-way, and adjacent land use (or area context) should be considered in lane and shoulder width decisions. Regardless of weather conditions, shoulders should be usable at all times. On high-volume highways, shoulders should preferably be paved, but paved shoulders may not always be practical. As a minimum, 2 ft [0.6 m] of the shoulder width should be paved to provide for pavement support, wide vehicles, and collision avoidance. Where bicycles are to be accommodated on the shoulder, a minimum paved width of 4 ft [1.2 m] should be used. The shoulder should be constructed to a uniform width for relatively long stretches of roadway. For additional information concerning shoulders, refer to Section 4.4.

Table 7-3. Minimum Width of Traveled Way and Usable Shoulder for Arterials in Rural Areas

U.S. Customary			
Design Speed (mph)	Minimum Width of Traveled Way (ft)[a] for Specified Design Volume (veh/day)		
	under 400[c]	400 to 2000	over 2000
40	20	22	24
45	20	22	24
50	22	22	24
55	22	24	24
60	22	24	24
65	22	24	24
70	22	24	24
75	22	24	24
All speeds	Width of Usable Shoulder (ft)[b]		
	4	6	8

Metric			
Design Speed (km/h)	Minimum Width of Traveled Way (m)[a] for Specified Design Volume (veh/day)		
	under 400[c]	400 to 2000	over 2000
60	6.0	6.6	7.2
70	6.0	6.6	7.2
80	6.6	6.6	7.2
90	6.6	7.2	7.2
100	6.6	7.2	7.2
110	6.6	7.2	7.2
120	6.6	7.2	7.2
130	6.6	7.2	7.2
All speeds	Width of Usable Shoulder (m)[b]		
	1.2	1.8	2.4

[a] On roadways to be reconstructed, an existing 22-ft [6.6-m] traveled way may be retained where the alignment is satisfactory and there is no crash pattern suggesting the need for widening.

[b] Preferably, usable shoulders on arterials in rural areas should be paved; however, where volumes are low or a narrow section is needed to reduce construction effects, the paved shoulder width may be a minimum of 2 ft [0.6 m] provided that bicycle use is not intended to be accommodated on the shoulder.

[c] Where frequent use by trucks is anticipated, additional traveled-way width should be considered.

7.2.3.2 Number of Lanes

The number of traffic lanes on an arterial in a rural area should be determined based on consideration of volume, level of service, context category (rural context or rural town context), and capacity conditions. A multilane arterial in a rural area, as discussed in this chapter, refers to an arterial facility with four or more total through lanes.

7.2.3.3 Cross Section and Right-of-Way

The type of surfacing and shoulder treatment should fit the volume and composition of motor-vehicle traffic and other modes, present or planned. Two-lane arterials in rural areas are normally crowned to drain away from the centerline except where superelevation is provided. Arterials in rural towns may have curb and gutter with inlet grates connected to underground stormwater collection systems. The treatment of cross slopes, drainage channels and systems, and side slopes is discussed in Chapter 4. The right-of-way is typically configured to accommodate all of the cross-sectional elements throughout the project. This usually precludes a uniform right-of-way width since there are typically many situations where additional width is advantageous. Such situations occur where off-street pedestrian and bicycle facilities are provided, where the side slopes extend beyond the normal right-of-way, for clear areas at the bottom of traversable slopes, for wide clear areas on the outside of curves, where greater sight distance is desirable, at intersections and junctions with highways, at railroad–highway grade crossings, for environmental considerations, and for maintenance access.

Local conditions, such as drainage, snow storage, presence of utilities, presence of freight, and presence of nonmotorized users should be considered in determining right-of-way widths. Where the need for additional lanes, shoulders, or roadside facilities is expected in the future for either motor-vehicle or nonmotorized users, the initial right-of-way width should be adequate to provide the wider roadway section. It may be desirable to construct the initial two lanes off center within the right-of-way, so the future construction will cause less interference with traffic and the investment in initial grading and surfacing can be salvaged.

7.2.4 Roadside Design

In the absence of roadside facilities for nonmotorized users, there are typically two primary considerations for roadside design along the traveled way for arterials in rural areas—clear zones and lateral offset.

7.2.4.1 Clear Zones

A clear unobstructed roadside is highly desirable on high-speed arterials in rural areas. Where fixed objects or non-traversable slopes fall within the clear roadside zones discussed in Section 4.6, "Roadside Design," refer to AASHTO's *Roadside Design Guide* (*6*) for guidance in selecting the appropriate treatment. Where practical, fixed objects, including trees that will grow to 4 in. [100 mm] or more in diameter, should be located near the right-of-way line and should be outside the selected clear zone. Where arterials in rural areas pass through a rural town context, the designer may refer to the "Arterials in Urban Areas" discussion in Section 7.3.4.

7.2.4.2 Lateral Offset

The full approach width (traveled way, shoulders, bicycle facilities, and sidewalks, if present) should be carried along the roadway and across bridges and overpasses where practical. To the extent practical, where another highway or railroad passes over the highway, the overpass should be designed so that the piers or abutment supports—including barrier systems—have a lateral offset no less than that of the approach roadway.

On facilities without curbing and with shoulder widths less than 4 ft [1.2 m], a minimum lateral offset of 4 ft [1.2 m] from the edge of the traveled way should be provided. Lateral offset is defined in Section 4.6.2. Further discussion and suggested guidance on the application of lateral offsets is provided in the AASHTO *Roadside Design Guide* (*6*).

7.2.5 Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the current *AASHTO LRFD Bridge Design Specifications* (*9*). The design loading should be the HL-93 calibrated live load designation.

The full width for the approach roadways, including shoulders and any space allocated for bicycles and pedestrians, should normally be continued across all new bridges. Long bridges, defined as bridges having an overall length in excess of 200 ft [60 m], may have a lesser width if current or projected bicycle use is very infrequent and no pedestrian facility is needed. On long bridges, shoulders should be at least 4 ft [1.2 m] measured from the edge of the traveled way on both sides of the roadway, and may need to be wider depending on existing and projected bicycle volumes. Where pedestrian facilities are provided, they must be accessible to and usable by individuals with disabilities (*37, 38*). See Section 10.8.3 for further information on bridge widths.

7.2.5.1 Vertical Clearances

New or reconstructed structures should provide 16-ft [4.9-m] clearance over the entire roadway width including the usable width of shoulders. Additional clearance to allow for future resurfacing should be considered. Existing structures that provide clearance of at least 14 ft [4.3 m], if allowed by local statute, may be retained. The vertical clearance to sign supports and to bicycle and pedestrian overpasses should be 1.0 ft [0.3 m] greater than the highway structure clearance.

7.2.6 Traffic Control Devices

Signs, pavement delineation, and pavement marking play an important role in the optimal operation of arterials in rural areas. Placement of these items should be considered early in the design stage while adjustments to the alignment and intersection design can be easily considered. Refer to the current MUTCD (*12*) for guidance in signing and marking.

7.2.7 Erosion Control

Consideration of erosion control features is important to the proper design of an arterial in a rural area. By controlling erosion, the design of the roadside is maintained and the environment downstream is protected from siltation and other possible harmful effects. Providing adequate ground treatment and cover has the additional benefit of assuring a pleasing roadside appearance.

7.2.8 Provision for Passing

In designing two-lane, two-way arterials in rural areas, the alignment and profile should normally provide sections suitable for passing at frequent intervals. Design of the horizontal and vertical alignment should provide adequate passing sight distance over as large a proportion of the highway length as practical. Table 7-1 presents the minimum passing sight distances for design speeds of 30 mph [50 km/h] and greater. Passing is not typically permitted on roadways with design speeds below 30 mph [50 km/h]. Restrictive cases may exist where passing sight distance is economically difficult to justify. Even in those instances, passing opportunities should be provided with at least the frequency needed to attain the desired level of service. Where achievement of sufficient passing sight distance is not practical, auxiliary lanes such as truck climbing lanes or passing lanes should be considered as a means to obtain the desired level of service.

Although truck climbing lanes are normally provided to prevent unreasonable reductions in operating speeds on upgrades, they also provide opportunities for passing in areas where passing would not otherwise be permitted. Adequately designed and well-marked climbing lanes will usually be used by slow-moving vehicles and allow passing by drivers who prefer to move at normal speeds. Climbing lanes are usually provided to the right of the through-traffic lane and should be the same width as the through lanes with a somewhat reduced shoulder width. A usable shoulder width of 4 ft [1.2 m] or greater is generally acceptable, although the existing or future presence of bicycles or pedestrians should be considered in the selection of narrower shoulders. The design elements and warrants for the use of climbing lanes are discussed in Section 3.4.3. An example of a climbing lane on a two-lane arterial in a rural area is shown in Figure 7-1.

Source: Oregon DOT

Figure 7-1. Climbing Lane on Two-Lane Arterial in a Rural Area

Passing lanes should be considered where climbing lanes are not warranted and where the extent and frequency of passing sections are too few. The use of passing lanes to increase passing opportunities on two-lane highways is addressed in Section 3.4.4.

In summary, the design procedures to be followed in providing passing opportunities on two-lane highways include:

- Design of the horizontal and vertical alignment should provide as great a proportion of the highway length as practical with adequate passing sight distance.
- For design volumes approaching capacity, the effect of passing opportunities on increasing capacity should be considered.
- For further information for climbing lane warrants, refer to Section 3.4.3.
- Where the extent and frequency of passing opportunities made available by application of items 1 and 3 are insufficient, the design should consider provision of passing lanes utilizing a three-lane cross section.

7.2.9 Ultimate Development of Multilane Divided Arterials in Rural Areas

Although many arterials in rural areas will adequately serve the traffic demands in the future, there are numerous instances where such arterials in rural areas will ultimately need development into a higher type arterial. Where an arterial in a rural area is being improved and it is anticipated that the DHV for the design year will exceed the service volume of the roadway for its desired level of service, the initial improvement should be consistent with the planned ultimate development, and acquisition of the needed right-of-way should be considered. This is particularly common in growing rural town contexts and emerging urban and suburban context areas, where changes in adjacent land use may significantly increase traffic volumes and presence of nonmotorized users. Ultimate roadway improvements may need enhanced pedestrian facilities, bicycle facilities, bus stops, transit or HOV lanes, interchanges and other features, in addition to travel lanes and shoulders. All of these needs should be considered.

For divided arterials in rural areas, the median should be widened to allow for the planned ultimate development to be added in the median. The initial two-lane arterial should be constructed so that it can eventually become one of the two two-lane, one-way roadways in the ultimate development of a four-lane divided arterial. The advantages of this approach are as follows:

1. There is no loss of investment in existing surfacing and overcrossings when the second roadway is constructed.
2. Traffic will be subjected to reduced restriction or delay when the additional two lanes are constructed because the original two lanes continue in use as a two-way arterial during construction, no detours are needed, and contact with construction operations is restricted to intersections and turnouts on one side.

3. Acquiring right-of-way with the initial improvement preserves right-of-way for the ultimate development. Acquiring right-of-way at current, rather than future, land values, particularly after the construction or improvement of the arterial, may more than offset the added initial right-of-way cost.
4. By grading the entire roadway for four lanes, future effects on wetlands created by roadside ditches and recharge basins are avoided, as well as erosion and other concerns associated with grading.

Care should be exercised, however, to provide an appropriate clear zone in the initial stage. A similar precaution may be adopted for top-soiling, seeding, planting, and any other work that is done to prevent soil erosion, steps which increase in value with time.

Two-lane arterials in rural areas planned for ultimate conversion to a divided arterial usually have sufficient initial volume to warrant a traveled way of 24 ft [7.2 m] wide and usable shoulders, 8 ft [2.4 m] wide, as shown in Figure 7-2A. These traveled way and shoulder dimensions are commensurate with those recommended for four-lane divided arterials in rural areas, as discussed in Section 7.2.11. For an arterial in a rural area that will ultimately be developed into a four-lane divided arterial having a wide median and an initial offset to one side of the right-of-way centerline, the roadway generally is crowned to drain both ways. Ultimately, a wide median should be depressed to be self-draining and may receive surface runoff from one-half of each roadway (Figure 7-2B). Grading for the future development generally is deferred when the median is wide.

Where the right-of-way for the future four-lane arterial in a rural area is restricted, a narrow median, which should be not less than 4 ft [1.2 m] wide, may need to be used. If provision of a median barrier is anticipated for the ultimate improvement, space for a wider median should be provided to accommodate the width of the barrier plus the appropriate clearance between the edge of the traveled way and the face of the barrier. As in the case of a wide median, the initial two-lane construction should be offset so that the ultimate development is centered on the right-of-way. To economize on the cost of drainage structures and simplify construction, the initial and future two-lane roadways may be positioned to drain to the outside (Figure 7-2C). It may be possible to defer future grading, depending on local conditions and on the probable length of time to the full development.

On many older two-lane arterials in rural areas, no provision was originally made for future improvement to a higher roadway type. In such instances, where practical, a new two-lane, one-way roadway should be provided approximately parallel to the first, which is then converted into one-way operation to form a divided arterial in a rural area. Where there is adjacent development, it may be more practical to construct another one-way, two-lane roadway some distance from the initial facility without disturbing the existing development. This method may also be advantageous where topography is not favorable to direct widening of the existing roadway section. If this method cannot be used, it may be practical to achieve a divided section by widening

14 ft [4.2 m] on each side of the existing roadway (Figure 7-2D). When none of these methods is practical, it may be appropriate to find a new location. The old road then becomes a local facility and may also serve as an alternate route. From the standpoint of adequacy and service provided to through traffic, the last method is preferred because the arterial in a rural area on a new location will not be influenced by the old facility and can be built to modern design criteria, preferably with some control of access.

For roadways that will ultimately be developed with narrow medians (Figures 7-2C and 7-2D), all of the cross sections shown in Figure 7-2 have minimum combined widths of roadways and median of 70 ft [20 m]. About 12 ft [3.6 m] or more of additional width should be obtained so that median lanes for left turns may be provided at intersections. Although the cross sections in Figure 7-2 do not show sidewalks, bicycle lanes, and shared-use paths, those facilities may be present in the ultimate development of some rural multilane arterials.

7.2.10 Multilane Undivided Arterials in Rural Areas

Research has shown that multilane undivided facilities often have substantially more collisions than multilane divided facilities with medians. Therefore, in new construction of multilane arterials, a median or central two-way left-turn lane should normally be provided. Multilane undivided arterials should be provided in new construction in rural areas only where provision of a median or central turn lane is not practical. A multilane undivided arterial in a rural area is the narrowest arterial in a rural area on which each traffic lane is intended to be used by traffic in one direction of travel, and all passing is accomplished on lanes not subject to use by opposing traffic. Because of the generally higher volumes, drivers on multilane arterials in rural areas are confronted with additional traffic friction—from opposing traffic, roadsides, and traffic in the same direction. Frequency of at-grade crossings has appreciable influence on crash frequency and traffic capacity. Turn lanes and adequate intersection sight distance can substantially reduce the frequency of crashes at intersections.

The elements of design discussed in preceding chapters are generally applicable to multilane undivided arterials in rural areas, except that passing sight distance is not essential. The sight distance that should be provided at all points is the stopping sight distance because passing can be accomplished without using an opposing traffic lane. In addition, intersection sight distance, as described in Section 9.5, should be provided at intersections.

If traffic volumes justify the construction of multilane arterials in rural areas where speeds are apt to be high, it is generally advisable to separate opposing traffic by a median. All arterials in rural areas on new locations that need four or more lanes should be designed with a median. Preferably a median should be provided in conjunction with widening of an existing two-lane arterial in a rural area into a multilane facility.

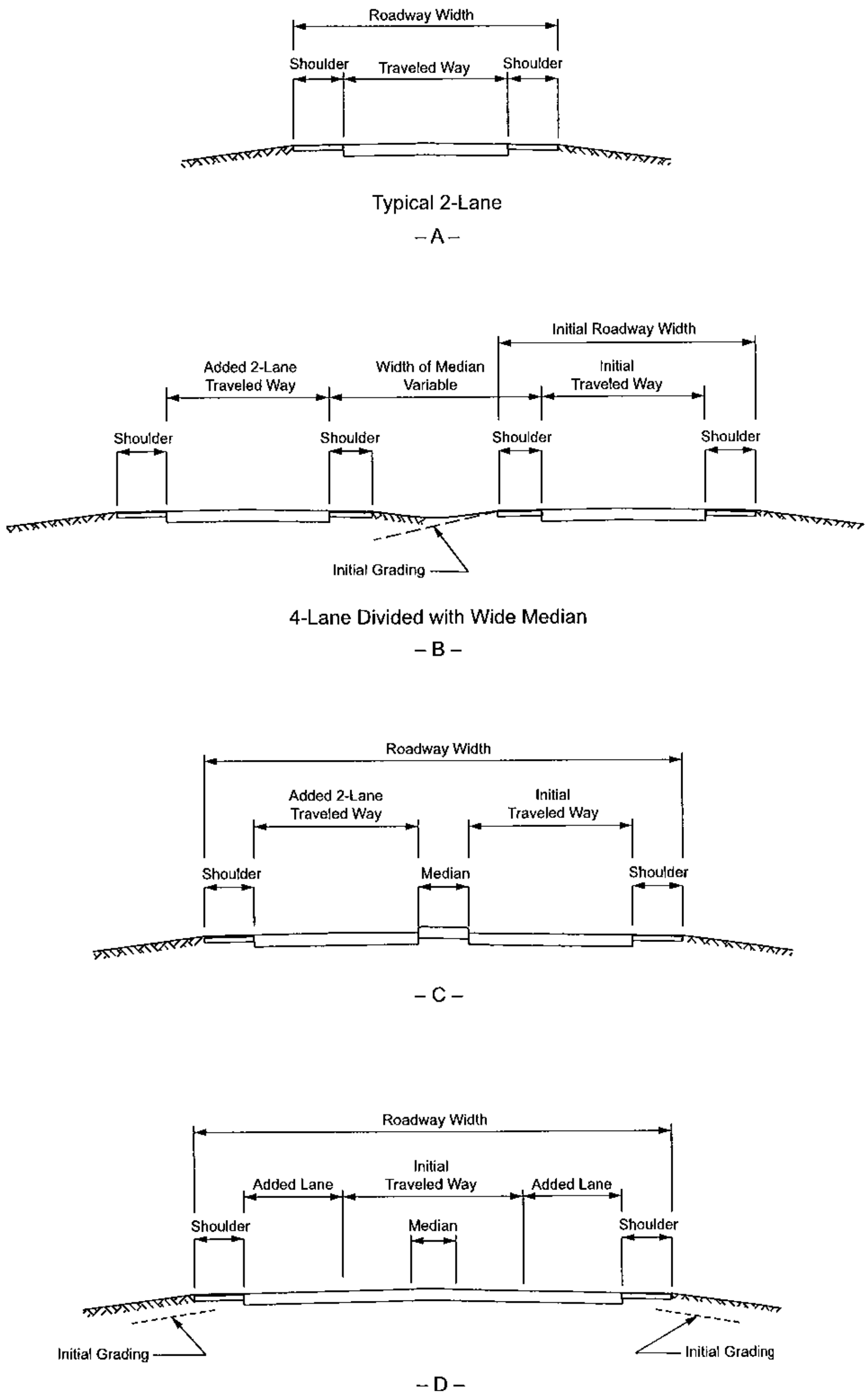


Figure 7-2. Two-Lane Arterial Cross Section with Ultimate Development to a Four-Lane Arterial

7.2.11 Divided Arterials in Rural Areas

7.2.11.1 General Features

A divided arterial in a rural area is one with separated lanes for traffic in opposite directions. It may be situated on a single roadbed or may consist of two widely separated roadways. The width of the median may vary and is influenced largely by the type of area, character of terrain, intersection treatment, and economics. An arterial in a rural area is not normally considered divided unless there are two full lanes in each direction of travel and the median has a width of 4 ft [1.2 m] or more and is constructed or marked to preclude its use by moving vehicles (except in emergencies or for left turns). A divided arterial in a rural area should have adequate median width to allow protected left turns, which can substantially reduce the frequency of crashes related to left-turn maneuvers.

The principal advantages of dividing multilane arterials in rural areas are reduced crash frequency, increased ease of operation, and increased comfort. A key reason for providing a median is to reduce head-on collisions, which are usually serious; such collisions may be virtually eliminated on roadways with wide medians or with a median barrier. Where median lanes for left turns are provided, this reduces rear-end collisions and impedance of through traffic resulting from left-turn movements. Pedestrians and bicyclists crossing the divided arterial in a rural area need to watch traffic in only one direction at a time and have a refuge at the median, particularly if a raised island is provided. Where the median is wide enough, crossing and left-turning vehicles can slow down or stop between the one-way roadways to take advantage of breaks in traffic and proceed when the driver decides it is safe to do so. Divided multilane arterials in rural areas provide more relaxed and pleasant travel than undivided arterials in rural areas, particularly in inclement weather and at night when headlight glare is bothersome. Headlight glare is reduced somewhat by addition of a narrow median, but it can almost be eliminated by addition of a wide median or a glare screen on a median barrier.

7.2.11.2 Lane Widths

Due to the high speeds and large volumes typically associated with divided arterials in rural areas, they should be designed with lanes 12 ft [3.6 m] wide. On reconstructed arterials in rural areas, it may be acceptable to retain 11ft [3.3-m] lanes if the alignment is satisfactory and there is no crash pattern suggesting the need for widening. In rural town contexts with low-speed conditions and low percentages of trucks, 10-ft lanes may be satisfactory.

7.2.11.3 Cross Slope

Each roadway of a divided arterial in a rural area may be sloped to drain to both edges, or each roadway may be sloped to drain to its outer edge, depending on climatic conditions and the width of median. Roadways on divided arterials in rural areas should have a normal cross slope of 1.5 to 2 percent.

When three or more lanes are inclined in the same direction on multilane divided arterials in rural areas, each successive pair of lanes outward from the first two lanes adjacent to the crown line may have an increased slope. A cross slope should not normally exceed 3 percent on tangent alignment, however. In no case should the cross slope of an outer or auxiliary lane, or both, be less than that of the adjacent lane.

For a more complete discussion, see Section 4.2.2, "Cross Slope."

7.2.11.4 Shoulders

Arterials in rural areas with sufficient traffic volume to justify the provision of four lanes will also justify having full-width shoulders. The width of usable outside shoulders should be at least 8 ft [2.4 m] and be usable during all seasons. Paving of the usable width of shoulder is preferred. Shoulders on arterials in rural areas are also desirable for use by bicyclists and occasional pedestrians. Where bicycles are to be accommodated on the shoulder, a minimum paved shoulder width of 4 ft [1.2 m] should be used with appropriate treatments for any rumble strips that may be added.

The normal roadway section, including usable shoulders, should be extended across all structures where practical. If the normal roadway section includes special accommodations for existing or anticipated pedestrian and bicycle users, then long bridges should also provide those cross section elements across their length.

Shoulder space on the left side of the individual roadways of a four-lane divided arterial in a rural area (i.e., within the median) does not serve the same purpose as the right shoulder. The shoulder on the right, through customary use on undivided arterials in rural areas, is understood by drivers as a suitable refuge space for stops and by nonmotorized users as available for their use. Where the median is flush with the roadway or has sloping curbs, vehicles may encroach or drive on it momentarily if forced to do so to avoid a crash. Only on rare occasions should drivers need to use the median for deliberate stops.

On divided arterials in rural areas with two lanes in each direction, a paved shoulder 4 ft [1.2 m] wide should satisfy the needs for a shoulder within the median. Such a shoulder will preclude rutting at the edge-of-traveled way and will reduce the likelihood of loss of control for vehicles that inadvertently encroach on the median.

On divided arterials in rural areas with three or more lanes in each direction, a driver in distress in the lane nearest the median may have difficulty maneuvering to the right-hand shoulder. Consequently, a full-width shoulder within the median is desirable on divided arterials in rural areas having six or more lanes.

Guardrail and median barrier should be considered in accordance with the AASHTO *Roadside Design Guide* (6).

7.2.11.5 Median Barrier Clearance

In cases where a wall or median barrier is used in the median, the AASHTO *Roadside Design Guide* (*6*) should be consulted for guidance in selecting an appropriate lateral clearance from the normal edge of the traveled way to the base of the wall or barrier and the type of barrier to be used.

7.2.11.6 Medians

On arterials in rural areas without at-grade intersections, the median may be as narrow as 4 to 6 ft [1.2 to 1.8 m] under very constrained conditions, but wider medians should be provided, wherever practical. A wide median allows the use of independent profiles. In addition, provision of a wide median may reduce the frequency of cross-median crashes and reduce headlight glare from vehicles in the opposing direction of travel.

Where intersections are to be provided, special concern should be given to median width. NCHRP Report 375 (*21*) found that most types of undesirable driving behavior in the median areas of divided highway intersections are associated with competition for space by vehicles traveling through the median in the same direction. The potential for such problems is reduced where crossroad and U-turn volumes are low, but may increase at higher volumes. Types of undesirable driving behavior observed include side-by-side queuing, angle stopping, and encroaching on the through lanes of a divided highway. At rural unsignalized intersections, the frequency of undesirable driving behavior and crashes was observed to decrease as the median width increased; this suggests that medians should be as wide as practical. It was also found that the frequency of undesirable driving behavior increased as the median opening length increased.

While medians as narrow as 4 to 6 ft [1.2 to 1.8 m] may be used under very restricted conditions, medians 12 to 30 ft [3.6 to 9 m] wide provide a protected storage area for left-turning vehicles at intersections. Medians of 4 to 8 ft [1.2 to 2.4 m] wide should be avoided, if practical, where left turns are common. Such widths do not provide sufficient space for turning vehicles and may encourage other motorists to encroach into the adjacent lane to avoid a turning vehicle that is only partially in the median. Where a median may be used as a pedestrian refuge, it should have a minimum width of 6 ft [1.8m].

In many cases, the median width at rural unsignalized intersections is a function of the design vehicle selected for turning and crossing maneuvers. Where a median width of 25 ft [7.5 m] or more is provided, a passenger car making a turning or crossing maneuver will have space to stop in the median area without encroaching on the through lanes. Medians less than 25 ft [7.5 m] wide should be avoided at rural intersections because drivers may be tempted to stop in the median with part of their vehicles unprotected from through traffic. The school bus is often the largest vehicle to use the median roadway frequently. The selection of a school bus as the design vehicle results in a median width of 50 ft [15 m]. Larger design vehicles, including trucks, may be used in the design of intersections where enough turning or crossing trucks are present; median widths of at least 100 ft [30 m] may be needed to accommodate large tractor–trailer trucks without encroaching on the through lanes of a major road.

For intersections with medians wider than 18 ft [5.4 m], it is desirable to offset any left-turn lanes provided to reduce sight restrictions due to opposing left-turn vehicles. Intersection designs with offset left-turn lanes are discussed in Section 9.7.3.

An intersection with a median so wide that drivers on the crossroad approach cannot readily see the far roadway of the divided highway may mislead some drivers into not recognizing the highway as divided. Such designs should be avoided where practical and, where they are used, signing and visual cues should be provided to discourage wrong-way movements.

Median widths over 60 ft [18 m] are undesirable at intersections that are signalized or may need signalization in the foreseeable future. The efficiency of signal operations decreases as the median width increases, because drivers need more time to traverse the median. Special detectors may be needed to avoid trapping drivers in the median at the end of the green phase for traffic movements across the median. Furthermore, if the median is so wide that separate signals are needed on each roadway of the divided highway, delays to motorists will increase substantially and attention should be given to vehicle storage needs on the median roadway between the two signals.

The discussion of median widths at intersections on arterials in urban areas in Section 7.3.3 indicates that wider medians may increase crashes and lead to undesirable driving behavior at intersections on arterials in urban areas. Therefore, consideration should be given to limiting use of wider medians at rural and rural town intersections that are likely to undergo urban or suburban development in the foreseeable future.

Undesirable driving behavior at rural unsignalized intersections increases as the median opening length increases (*21*). The median opening length should be equal to at least that described in Section 9.8, but median openings at rural unsignalized intersections should not be unnecessarily long. For additional guidance, refer to NCHRP Report 633, *Impact of Shoulder Width and Median Width on Safety* (*29*).

Medians should be designed to provide a forgiving roadside. Guardrail or median barrier should be considered in accordance with the AASHTO *Roadside Design Guide* (*6*). Further information on median design is presented in Section 4.11.

7.2.11.7 Alignment and Profile

A divided arterial in a rural area generally serves high-volume and high-speed traffic for which a smooth flowing alignment should be provided. Because a divided arterial in a rural area consists of two separated roadways, there may be instances where median widths and roadway elevations can be varied. Special topographic or intersection considerations may make such treatments desirable for economic or operational reasons. Precaution should be taken so that such variations do not adversely affect operations. Potential problems associated with sharp reverse curves, headlight glare, roadside design, sight distance, and grades of intersection crossings should be considered.

Profile design is less difficult for multilane arterials than for two-lane arterials in rural areas. With two or more lanes for travel in each direction, the profile grade is generally governed by stopping sight distance, except at intersections. For volumes well below capacity, grades may be steeper and longer on multilane arterials than on two-lane arterials in rural areas, because there is a continuous lane for passing of heavy, slow vehicles on upgrades.

Although vertical design controls may be less restrictive for divided arterials than for two-lane arterials because passing sight distance need not be considered, the design of appropriate profiles for divided arterials involves design judgment and careful study. Even though a profile may satisfy all of the design controls, the finished product can appear forced and angular. A smoothly-flowing roadway with gradual changes in horizontal and vertical alignment should be designed to the extent practical. Such design is of primary importance where a median of constant width is used in rolling terrain. The lack of a need to provide passing sight distance may tempt designers to use a roller coaster profile, which appears more displeasing on a divided arterial than on a two-lane arterial in a rural area. With a wide divided arterial of uniform cross section, the driver's longitudinal perspective of distance is compressed and can make the combination of horizontal and vertical alignment appear abrupt and disjointed. The relationship of horizontal and vertical alignment should be studied to obtain a suitable combination. To avoid an undesirable appearance, profile designs should be checked in long continuous plots, wherein the foreshortened aspect can be simulated. Section 3.5, "Combinations of Horizontal and Vertical Alignment", provides additional guidance on this topic.

7.2.11.8 Climbing Lanes on Multilane Arterials in Rural Areas

Multilane arterials in rural areas usually have sufficient capacity to handle their traffic load, including the normal percentage of heavy trucks, without becoming severely congested. Climbing lanes generally are not as easily justified on multilane arterials as on two-lane arterials, because on two-lane arterials drivers following slow-moving trucks on upgrades may be unable to or psychologically discouraged from using an adjacent traffic lane for passing. By contrast, on multilane arterials, drivers have an adjacent lane available to them in which to pass slow-moving vehicles.

In addition, a climbing lane on a two-lane, two-way arterial in a rural area is useful during both peak and non-peak hours, whereas on a multilane arterial in a rural area, a climbing lane is likely to have only limited use during non-peak hours. During periods of lower traffic volumes, a vehicle following a slow-moving truck in the right lane can readily move to the adjacent lane and proceed without difficulty, although there is evidence that slow vehicles on through-traffic lanes may cause crashes.

Because new or reconstructed arterials are designed for 20 years or more in the future, there is little likelihood of climbing lanes being justified on multilane arterials for several years after their initial construction, even though climbing lanes are deemed desirable for the peak hours of the design year. Thus, there may be an economic advantage in designing for, but deferring construction of, climbing lanes on multilane arterials. In this situation, grading for the future climbing lane should be provided initially. Very little additional grading is needed because a full shoulder is likely to be provided where there is no climbing lane; however, only a narrow shoulder is typically used outside of a climbing lane, because the climbing lane itself can serve as an emergency stopping area when needed. A full discussion on the need for climbing lanes is found in Section 3.4.3.

7.2.11.9 Superelevated Cross Sections

A divided arterial in a rural area on a curve should typically be superelevated to enhance vehicle control and offer a pleasing appearance. Care should be taken in the superelevation transition to fit site conditions and to meet controls of intersection design.

General methods of attaining superelevated cross sections for divided arterials in rural areas are discussed in Section 3.3.8.6. In the design of arterials in rural areas, the inclusion of a median in the cross section alters the manner in which superelevation is attained. Depending on the width of median and its cross section, there are three general cases for attaining superelevation.

Case I—The whole of the traveled way, including the median, is superelevated as a plane section. Case I should be limited to narrow medians and moderate superelevation rates to avoid substantial differences in elevation of the extreme edges of the traveled way arising from the median tilt. Specifically, Case I should be applied only to medians with widths of 15 ft [4.5 m] or less.

Case II—The median is held in a horizontal plane and the two traveled ways are rotated separately around their median edges. Case II can apply to any width of median but is most appropriate for medians with widths between 15 and 60 ft [4 and 18 m]. By holding the median edges level, the difference in elevation between the extreme traveled-way edges can be limited to that needed to superelevate the roadway. Superelevation transition design for Case II usually has the median-edge profiles as the control. One traveled way is rotated about its lower edge and the other about its higher edge.

Case III—The two traveled ways are treated separately for superelevation with a resulting variable difference in elevation at the median edges. Case III design can be used on wide medians (i.e., those with widths of 60 ft [18 m] or more). For this case, the difference in elevation of the extreme edges of the traveled way is minimized by a compensating slope across the median. With a wide median, it is possible to design the profiles and superelevation transition separately for the two roadways.

Section 3.3.8, particularly Figure 3-8, contains additional guidance concerning methods for attaining superelevation for Cases I, II, and III.

Figure 7-3 shows the treatment of cross sections for superelevated roadways with narrow and wide medians in relation to the width of median for the three cases noted. In the cross sections shown in Figures 7-3A and 7-3D, both roadways lie in the same plane. The roadways are superelevated by rotating them about a profile control on the centerline of the median. The same effect can be obtained by rotation about the edge of the traveled way or any other convenient control line.

Where the cross section shown in Figure 7-3A is used, the median should be graded in accordance with the *AASHTO Roadside Design Guide* (*6*) and designed so that surface water from the higher roadway does not drain across the lower roadway. On tangent alignment, a shallow drainage swale can be provided in a median about 15 ft [4.5 m] wide and a well-rounded drainage channel with a width of about 60 ft [18 m] as shown in Figure 7-3F. On a superelevated section rotated about the median centerline, as in the cross section shown in Figure 7-3A, approximately 30 ft [9 m] of median width is needed for a rounded drainage channel and adequate left shoulders. In a median less than 30 ft [9 m] wide, a channel with flat sideslopes can be provided if the superelevation rate is small, or a paved channel can be used in conjunction with higher rates of superelevation.

The projection of superelevation across wide medians may be fitting in some instances, as in the cross section shown in Figure 7-3A, but its general use in conjunction with large rates of superelevation is not satisfactory in appearance and generally not economical. It may fit at highway intersections where the profile of the intersecting road approximates the superelevated slope. Occasionally, it may fit the natural slope of the terrain. However, unless these conditions prevail, the large difference in elevation between the outer shoulder edges is likely to be objectionable. For example, the difference in elevation between the outer shoulder edges of a four-lane divided arterial in a rural area with a median of 40 ft [12 m] and a superelevation rate of 8 percent is about 8 ft [2.4 m].

In level terrain and in terrain where the natural slope of the land is adverse to the cross-sectional slope, substantial improvement in appearance and economy in earthwork results if the wide median is level as in the cross section shown in Figure 7-3B, or sloped opposite to the superelevation plane as shown in Figure 7-3C.

Superelevation runoff lengths may vary for each of the three cases (refer to Table 3-16). For Case I designs, the length of runoff should be based on the total rotated width (including the median width). Runoff lengths for Case II designs should be the same as those for undivided highways with a similar number of lanes. Finally, runoff lengths for Case III designs are based on the needs of the separate one-way roadways, as defined by their superelevation rates and rotated widths.

In the cross sections shown in Figures 7-3B and 7-3E, the edges of the roadways on the median sides are at the same elevation. Designs on this basis are pleasing in appearance and generally operate effectively. With a wide separation between the one-way roadways, the cross section shown in Figure 7-3B has considerable advantage over that shown in Figure 7-3A in the reduction in difference in elevation across the entire roadbed. On roadways having a superelevation rate near 10 percent, the treatment shown in

Figure 7-3B needs a minimum median width of about 30 ft [9 m] to provide fully effective shoulder areas and a well-rounded traversable swale.

In the cross sections shown in Figures 7-3C and 7-3F, the two one-way roadways have a common centerline grade. The difference in elevation of the outer extremities of the superelevated roadways is minimal, being the product of the superelevation rate and the width of one of the one-way roadways. The method of attaining superelevation runoff is directly applicable to each one-way roadway.

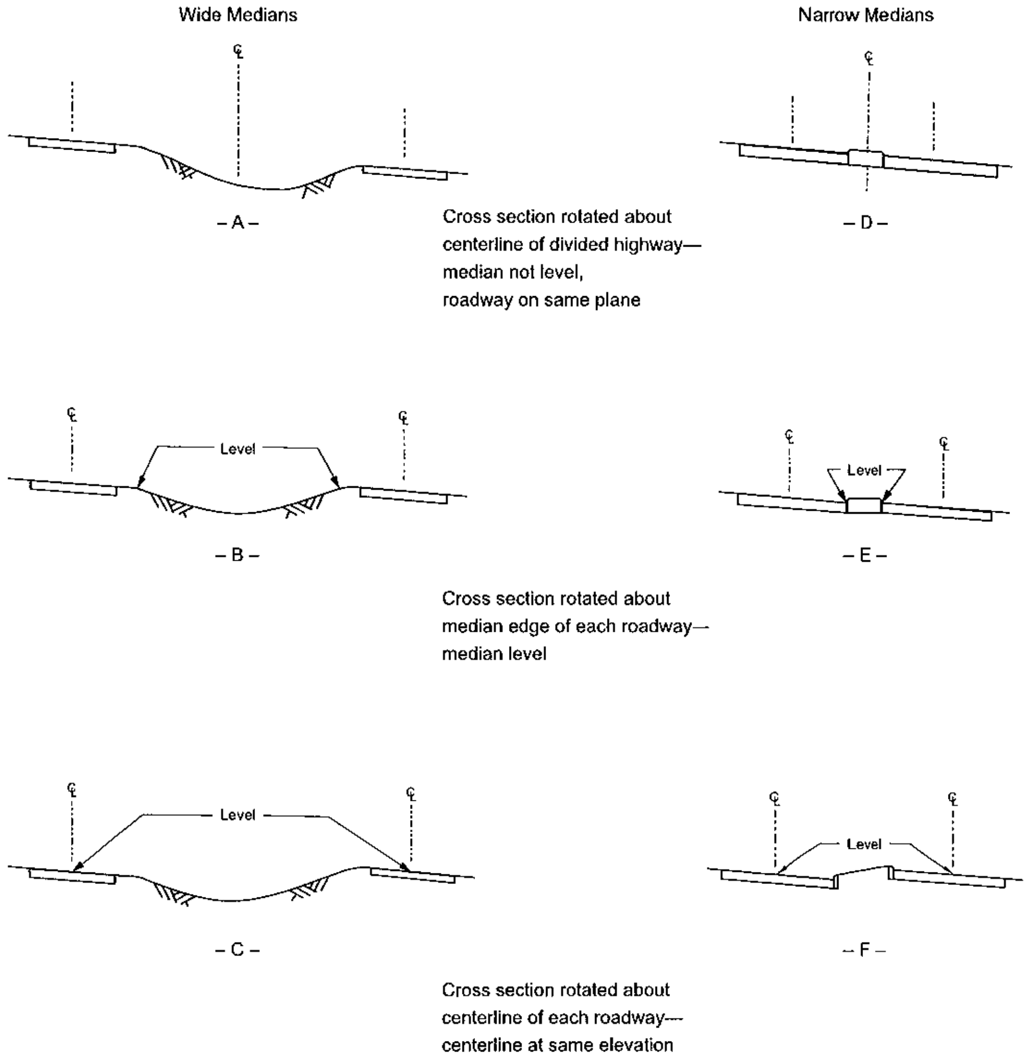


Figure 7-3. Methods of Attaining Superelevation on Divided Arterials in Rural Areas

With a wide median, the treatment shown in Figure 7-3C allows the desired appearance to be maintained and permits economy in the wide-graded cross section. The roadway as a whole will appear fairly level to the motorist, who will not readily perceive the difference in elevation of the inside edges of roadway. This cross section generally is not suitable for important at-grade intersections unless the median is very wide. The median should be sufficiently wide in relation to superelevation to provide a smooth S-shaped profile across its width. The width for this shape is somewhat more than that needed for the previous sections. About 40 ft [12 m] is needed, with a superelevation rate of 10 percent and adequate shoulder areas. This width can be reduced to about 30 ft [9 m] when a paved channel is provided.

On a divided arterial in a rural area with variable width of median and difference in elevations for the two roadways, each roadway is designed with a separate profile. With a reasonably wide median, each roadway can be superelevated in any manner suitable for a single roadway with little effect on the median slope. A retaining wall may be needed in a narrow median if an appreciable difference in elevation exists. The manner of superelevating the roadways has some effect on the height of the wall, but this amount is minimal and should have little bearing on design. Figure 7-4 shows various median configurations that may be used on arterials in rural areas. The configurations shown in Figures 7-4A, 7-4F, and 7-4G are appropriate for rural settings, while the configurations shown in Figures 7-4C, 7-4D, and 7-4E are more appropriate for urban situations as described in Section 7.3. The configuration in Figure 7-4B may be used in either setting. Refer to the AASHTO *Roadside Design Guide* (*6*) for guidance on designing a forgiving roadside.

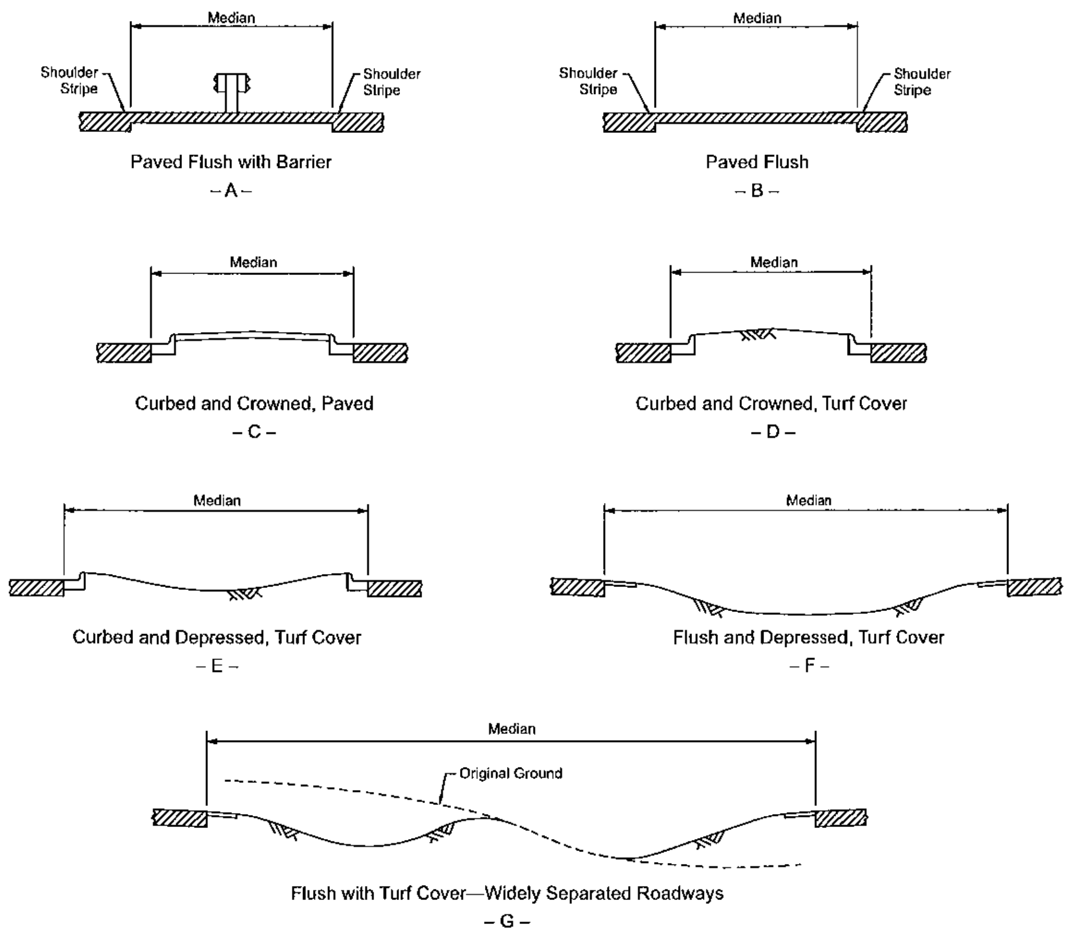


Figure 7-4. Typical Medians on Divided Arterials

7.2.11.10 Cross Section and Right-of-Way Widths

Cross-sectional elements of divided arterials in rural areas—the widths and details of traveled ways, shoulders, medians, sideslopes, clear zones, and drainage channels—have been discussed separately in this and other chapters. The appropriate right-of-way widths, including typical elements in a composite arterial cross section, are presented in Figure 7-5. Nontypical elements and intermittent features such as auxiliary turn lanes may also control right-of-way needs and should be taken into consideration. As arterials in rural areas approach and pass through a rural town context, the cross section may also need to incorporate elements for other users such as pedestrians, bicyclists, and on-street parking. Refer to Section 7.2.19 for a discussion on transitioning from high-speed to low-speed arterials.

In an ideal situation, the topography, other physical constraints, and economic feasibility permit the design of a well-balanced cross section of desirable dimensions, for which an adequate width of right-of-way is established and procured. On the other hand, the constraints may be so tight that if a divided arterial in a rural area is to be provided at all, it should be designed within a limited width of right-of-way, using minimum or near-minimum dimensions for each element of the arterial cross section. In the first instance, the right-of-way is based on the most favorable design criteria for the cross-sectional elements; in the latter case, the cross section is determined on the basis of the available width of right-of-way.

The widths of cross-sectional elements should be proportioned to provide a well-balanced arterial section. Recommended traveled way and shoulder widths are shown in Table 7-3. The border width is affected directly by the depth of cut or fill. If the right-of-way is restricted, the border area or median width, rather than the lane or shoulder width, should be reduced. The extent to which the border area or median width, or both, is reduced respectively should be carefully decided. Providing a median width greater than that which eliminates the need for a median barrier is generally not warranted if doing so would subsequently involve installing substantial amounts of roadside guardrail that would otherwise not be needed, or if adjacent roadside development is present or anticipated. Consideration should be given to achieving approximately the same clear zone width for both the median and roadside.

Figure 7-5C shows a desirable divided arterial cross section warranted for a high-type facility where liberal width of right-of-way is attainable and bicycle/pedestrian accommodation is desired. Where these wider widths cannot be obtained, providing a right-of-way width that incorporates a median width of 30 ft [9 m] or more and sufficient borders to provide for the appropriate clear zone is desirable. For additional information on clear zones, refer to the AASHTO *Roadside Design Guide* (*6*).

Sometimes the right-of-way may be so restricted that minimum or near-minimum widths of cross-sectional elements need to be used. If at all practical, the right-of-way should be wide enough to permit the use of median and borders of not less than 15 ft [4.5 m] (see Figure 7-5A). A 15-ft [4.5-m] median is near the minimum median width within which a median lane can be provided at intersections. Figure 7-4 shows some sections with curbs, which are generally not recommended along rural roadways. Sloping curbs may be used in restricted areas where needed to control drainage, or where special treatment is needed at locations such as intersections.

The cross sections and right-of-way widths shown in Figure 7-5 pertain to four-lane facilities. If ultimate conversion to a six- or eight-lane facility is planned, the right-of-way widths should be increased by the width of lanes to be added. It is preferable to include this additional width in the median.

The cross-sectional arrangements shown in Figure 7-5 indicate generally balanced sections for what are termed "desirable," "minimum," and "restricted" rights-of-way. Some variation in these arrangements may be appropriate in individual cases. The right-of-way width need not be uniform and may be varied along the course of the arterial as needed for grading, for appropriate roadside design, and other conditions. Where substantial constraints are present, the two roadways may need to be brought closer together. Where physical conditions are favorable and land is readily available, the roadways of a divided highway may be spread farther apart. Where future grade separations and ramps are envisioned, consider initial acquisition of additional rights-of-way.

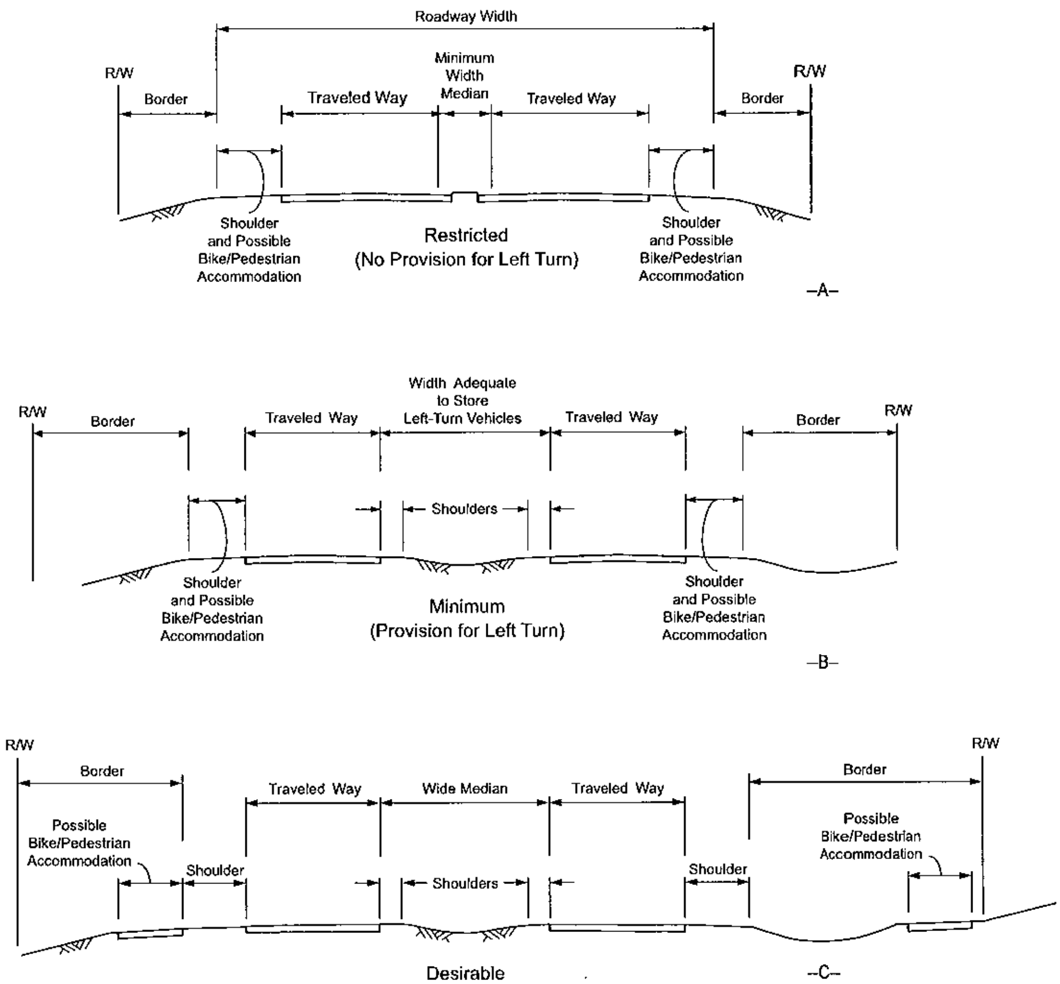
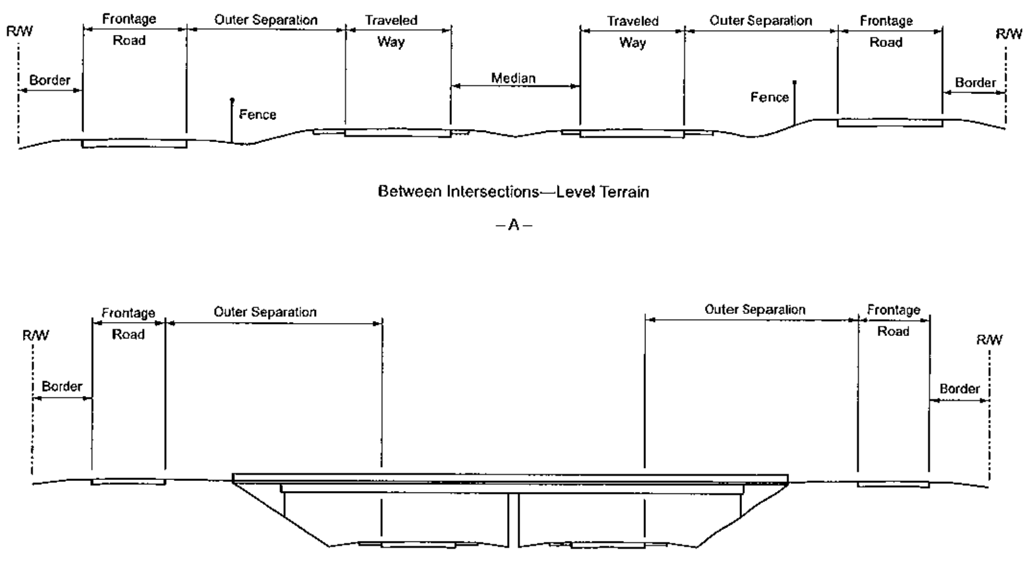


Figure 7-5. Cross Sectional Arrangements on Divided Arterials in Rural Areas

The cross sections depicted in Figure 7-5 represent normally divided facilities in rural areas. Sometimes in rural areas, and particularly in and near urban districts, it is appropriate to separate through traffic from local traffic. Where such is the case, frontage roads may be provided along the outer limits of the highway cross section (Figure 7-6). Frontage roads serve to collect and distribute local traffic to and from adjacent development and provide parking and service thereto removed from the main traveled way, thus freeing through traffic from the disturbance introduced by local operation. The component parts of a typical cross section with frontage roads in generally flat terrain are shown in Figure 7-6A. The frontage roads are shown within the right-of-way limits, which is the typical arrangement. Frontage roads sometimes are provided outside the right-of-way limits, in which case the right-of-way can be narrower than shown. Where the profile of the through-traveled way passes over or cuts through the natural ground, frontage roads are generally held at the level of the existing development, and the difference in elevation between the main traveled ways and the frontage roads is attained within the outer separations by earth slopes or retaining walls.



At Separated Crossroads

-B-

Figure 7-6. Cross Sectional Arrangements on Divided Arterials with Frontage Roads

Some crossroads in divided arterials in rural areas may be grade separated from the through-traveled way with local service provided by frontage or other roads. If all crossroads were grade separated in this manner, the facility would be a freeway. However, grade separation on divided arterials in rural areas may be appropriate at some crossroads but not at others. A typical cross section at a separated crossroad with a depressed arterial is depicted in Figure 7-6B. Where frontage roads are provided, the outer separations should be wider on arterials having two-way frontage roads and on arterials with grade separations than on arterials crossing at grade to allow for roadside slopes and ramps. Further discussion on interchanges is presented in Chapter 10.

7.2.11.11 Sections with Widely Separated Roadways

Occasionally it is advantageous to widely separate the one-way roadways of a divided arterial in a rural area. Widely separated roadways may be particularly appropriate for certain topographic conditions. In valleys where drainage makes the location difficult, individual roadways may be situated on each side of the valley. Drainage of roadways is then simplified, with both sides draining directly to the natural channel. Along ridges or where there is a continual change in ground cross slopes, the separate roadways may be better fitted to the terrain than an arterial on a single roadbed. Such arrangements simplify location problems because only one roadway is considered at a time. With reduced roadway prisms, construction scars are kept to a minimum and more of the natural growth is retained, particularly between the separate roadways. In areas where right-of-way is not restricted, designs involving widely separated roadways often result in lower construction costs.

A wide median design may be appropriate where an existing two-lane arterial in a rural area is improved to a four-lane section but direct widening is not practical because of topography or adjacent development. In such a case, the old roadway is not disturbed but is converted to one-way operation and another, completely separate, one-way roadway is constructed. This action sometimes results in acquisition of two separate rights-of-way to contain the individual roadways of the divided arterial in a rural area.

Intersections between a crossroad and a one-way roadway are simpler in design and operation than intersections between a crossroad and a two-way roadway. If designed properly, crash potential is generally reduced and the capacity of intersections is increased. Moreover, operation on widely separated roadways provides the maximum in driver comfort. Strain is lessened by largely eliminating the view and influence of opposing traffic. Substantial reduction or elimination of headlight glare at night is especially helpful in easing driver tension.

Operational problems of intersections on roadways with very wide medians should be considered. Desirably, a wide median is adequate to store the longest legal vehicles. To determine the number of intersection lanes needed, all movements and their volumes should be considered. The need for turnarounds, connecting roadways, and frontage roads should be considered along with the effect on adjacent property owners. Signing to prevent wrong-way operation should be provided in accordance with the MUTCD (*12*), particularly when both roadways of the divided highway are not visible to drivers stopped at the crossroad. Additional discussion on wide medians is also presented in Section 7.2.11.6, "Medians."

If arterials of appreciable length have roadways separated so widely that each roadway cannot be seen from the other, drivers may believe that they are on a two-way instead of a one-way roadway and hesitate to pass slow-moving vehicles. This situation can be alleviated by an occasional open view between the two roadways.

7.2.12 Intersections

The liberal use of interchanges and robustly-designed intersections is highly desirable on arterials in rural areas that do not have full control of access. Auxiliary turning lanes and adequate turning widths should generally be provided where arterials intersect with other public roads. Where practical, principal arterials in rural areas that intersect should ideally be served by interchanges, possibly of the free-flow type. A comprehensive study of each intersection is needed for new and reconstruction projects, and a suitable design, consistent with the desired level of service, should be selected.

Rural intersection control by traffic signals is normally not desirable. Drivers generally do not anticipate signals in rural areas or facilities with high operating speeds, especially when traffic volumes are relatively low. Curbed islands present an obstacle to drivers and may become snow traps in regions that receive frequent snowfalls. Therefore, curbs should be used sparingly at intersections in high-speed areas.

If interchanges are intermixed with intersections, adequate merging distances should be provided to allow ramp traffic to operate freely. The merging driver should not have to be concerned with cross traffic at a downstream intersection while making a merging maneuver. Design of intersections and interchanges should be in accordance with Chapters 9 and 10, respectively.

7.2.13 Access Management

Arterials in rural areas are designed and built with the intent of providing better traffic service than is available on local and collector roads and streets. Although an arterial in a rural area may not have more traffic lanes, its ability to carry greater volumes is usually related to the amount of crossroad interference or side friction to which it is subjected. One of the most important considerations in arterial development is the amount of access control, full or partial, that can be acquired. Effective control of access on an arterial in a rural area will often reduce the frequency of access-related crashes.

Controlling access is vital to preserving the level of service for which the arterial was initially designed. Access control is usually not too difficult to obtain in a rural area where development is light. Adequate access can normally be provided without great interference to traffic operations. However, rural areas do pose distinct access-related problems. The movement of large, slow-moving farm machinery is not uncommon and numerous field entrances are also requested by landowners. Because of these unique problems, access points should be situated to minimize their detrimental effects to through traffic. If access points are needed on opposite sides of the roadway, they should be situated directly opposite one another to reduce the time needed for vehicles to cross the arterial. Where access is needed for two adjacent properties or where different land uses adjoin one another, providing one driveway to serve both properties will reduce the number of access locations needed. Adequate and uniform spacing between access points will also help eliminate many conditions where a large vehicle at an intersection hides another vehicle on a nearby approach. Consideration should also be given to the location of access points in relationship to intersection sight distance restrictions and other intersections. High-volume access points can lead to particular operational problems if not properly situated. Short sections of rural frontage roads may be used to combine access points and minimize their operational effect to the arterial in a rural area.

The appropriate degree of access control or access management depends on the type and importance of an arterial in a rural area. Anticipation of future land use is a critical factor in determining the degree of access control. Provision of access management is vital to the concept of an arterial route if it is to provide the service life for which it is designed. For additional guidance on access management techniques for arterials in rural areas, refer to NCHRP Report 420, *Impacts of Access-Management Techniques* (*19*), and the TRB *Access Management Manual* (*34*).

7.2.14 Bicycle and Pedestrian Facilities

Arterials in rural areas often provide the only direct connection between populated areas and locations to which the public wishes to travel. Schools, parks, and rural housing developments are usually located to be readily accessible by automobile. However, pedestrians and bicyclists may also wish to travel to the same destination points, especially where arterials in rural areas pass through a rural town context or through a recreational area. Where demand for pedestrian and bicycle travel exists, or is expected due to planned changes in land use, the designer should consider the needs of pedestrians and bicyclists and provide pedestrian and bicycle facilities where appropriate.

On some roads with very low pedestrian and bicycle demand, paved shoulders may be an appropriate treatment. Where frequent pedestrian activity exists or is anticipated, pedestrians may be accommodated by sidewalks on one or both sides of the roadway; sidewalks must be accessible to and usable by individuals with disabilities (*37, 38*). Additional guidance is available in the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*36*). In addition, the AASHTO *Guide for the Planning, Design and Operation of Pedestrian Facilities* (*3*) presents appropriate methods for accommodating pedestrians, which vary among roadway and facility types, and provides guidance on the planning, design, and operation of pedestrian facilities. If on-street or off-roadway bicycle facilities are considered appropriate to meet current or future demand, they should be designed in accordance with the AASHTO *Guide for the Development of Bicycle Facilities* (*7*).

7.2.15 Bus Turnouts

Where bus routes are located on an arterial in a rural area, provision should be made for loading and unloading of passengers. Because of its size, a bus cannot easily leave the roadway unless special provisions are made. A well-marked, widened shoulder or an independent turnout is highly desirable and should be provided, if practical. Although it may be impossible or impractical to provide, for example, school bus turnouts for every dwelling, they should be provided at locations where there are known concentrations of passengers. Facilities to provide access to bus stops may also need to be provided from nearby destinations. Appropriate provisions for buses may provide greater capacity and reduced crash frequencies for an arterial in a rural area. For additional guidance concerning bus turnouts and access to bus stops, refer to Section 4.19 and the AASHTO *Guide for Geometric Design of Transit Facilities on Highways and Streets* (*8*).

7.2.16 Railroad–Highway Grade Crossings

Desirably, all railroad crossings on the system of arterials in rural areas should be grade separated. However, practical considerations make it likely that many crossings will be at grade. Various treatments can be applied at railroad–highway grade crossings to reduce the likelihood of crashes including adequate signing, lighting, signals, signals with gates, and grade separations. Judgment should be used in selecting appropriate design and traffic control treatments for railroad–highway crossings; factors to be considered include the volume and speed of traffic on both roadways and railroads, the mix of users and modes of travel, the available sight distance, and the anticipated crash reduction benefits of specific treatments. Given the high traffic volumes and speeds on many arterials in rural areas, and the severity of train-vehicle collisions, the designer should strive for the most protection that is practical. For further guidance on traffic control systems for railroad–highway grade crossings, refer to the MUTCD (*12*). For further information on design criteria for railroad–highway grade crossings, see Section 9.12.

7.2.17 Lighting

Adequate lighting can be important to reduce the potential for crashes on selected arterials in rural areas at night and can also aid older drivers. The higher speeds that are typically found on an arterial in a rural area make it especially challenging for the driver to make correct decisions with adequate time to execute the proper maneuvers without creating undue conflict in the traveled way. Most modern arterials in rural areas are designed with an open cross section and horizontal and vertical alignment of a fairly high type and, therefore, offer an opportunity for near maximum use of vehicle headlights, resulting in reduced justification for fixed highway lighting. In practice, the lighting of arterials in rural areas is seldom applied, except in the rural town context and in certain critical areas, such as interchanges, intersections, railroad grade crossings, long or narrow bridges, tunnels, sharp curves, and areas where roadside interferences are present.

Whether or not at-grade intersections in rural areas should be lighted depends on the adjacent land use, the layout of the intersection, and the traffic volumes involved for all modes. Intersections that do not have channelization are frequently left unlighted. On the other hand, intersections with substantial channelization, particularly multi-road layouts and those designed on a broad scale, are often lighted. It is especially desirable to illuminate large-scale channelized intersections and all roundabouts. The AASHTO *Roadway Lighting Design Guide* (*5*) and ANSI/IESNA RP-8 *American National Standard Practice for Roadway Lighting* (*30*) are recommended as sources of lighting information.

7.2.18 Rest Areas

The provision of rest areas on the system of arterials in rural areas is a desirable feature, particularly on principal arterials in rural areas. Rest areas provide the high-speed, long-distance traveler with the opportunity for short periods of relaxation, which relieves driver fatigue. These facilities serve the needs of motorists, as evidenced by public recognition of the issue of driver fatigue, as well as by the extensive use of rest areas.

The location of rest areas should be considered early in development of a multilane arterial in a rural area. Sites of special interest or visual quality provide additional reasons for the motorist to stop and often extend the length of their stay. The spacing of rest areas depends on many considerations. For example, construction and operating costs for rest areas are significant, but benefits to drivers should also be considered. Additional information on rest areas may be found in AASHTO's *A Guide for the Development of Rest Areas on Major Arterials and Freeways* (*1*).

7.2.19 Speed Transitions Entering Rural Towns

Rural arterials provide important connections to and through many rural towns. Where a high-speed rural arterial leaves the rural context and enters a rural town or other developed area, there will be a high-speed to low-speed transition zone within which drivers should reduce their speed to a speed consistent with the rural town environment. The transition area should be effectively designed to encourage speed reduction because, if drivers do not appropriately reduce speeds, they may create conflicts with other vehicles, pedestrians, and bicyclists and may adversely affect community livability. Design treatments that may be implemented, where appropriate, so that high-speed to low-speed transition zones function more effectively include: center islands, raised medians, roundabouts, roadway narrowing, lane reductions, transverse pavement markings, colored pavements, and layered landscaping. The treatments, alone or in combination, encourage drivers to reduce speeds by introducing a changed driving environment in which lower speeds appear appropriate to the driver.

A transition area consists of three elements—the rural context, the transition zone, and the rural town context, as shown in Figure 7-7.

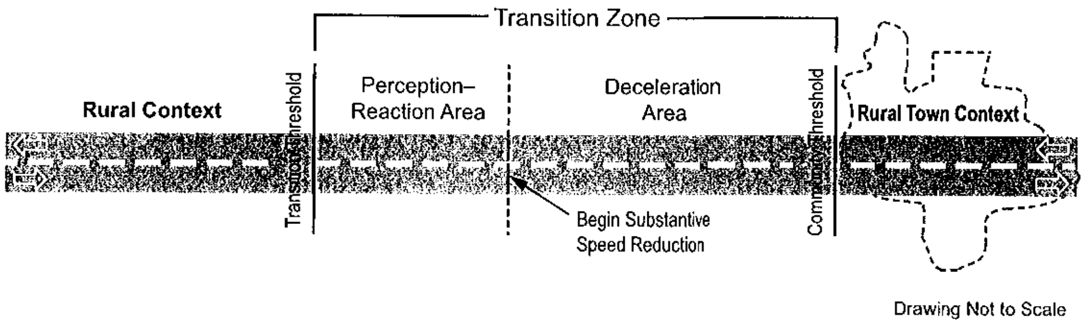


Figure 7-7. Transition Zone Areas (*31*)

The rural context typically has little roadside development and few access points, and arterials in the rural context are designed to facilitate high-speed, longer-distance travel. The transition zone includes two areas—a perception–reaction area and a deceleration area. It should have elements that differentiate it from its two abutting contexts—rural context and rural town context—and inform and assist drivers in making the appropriate speed reduction. The rural town context typically has lower design speeds, increased traffic control, on-street parking, sidewalks, curbs and gutters, higher land-use intensity, frequent access points, landscaping, street-trees, pedestrian and bicycle activity, narrow lanes, and turn lanes.

Transition zones should be designed to fit the characteristics of the roadway and the community. Guiding principles for the design of transition zones are:

- More extensive and aggressive treatments tend to produce greater reductions in speed and crash occurrence than less extensive and passive treatments.
- There needs to be a distinct relationship between the rural town speed limit and a change in the roadway character. Emphasizing a change in the environment increases driver awareness.
- Physical changes to the roadway and roadside are favored treatments because they have permanent and lasting effects. The effects of enforcement and education programs are more transient and less effective.
- Each transition zone and rural town has its own unique characteristics. As such, no particular treatment is appropriate for all situations. Each transition zone and rural town should be assessed on a case by case basis before selecting a treatment or combinations of treatments.
- Before selecting a treatment, consideration should be given to the two areas that make up the transition zone. In the perception–reaction area, warning and/or psychological treatments are appropriate, while in the deceleration area physical treatments should be installed.
- Combinations of treatments are more effective at reducing speeds and crashes within a transition zone and through a rural town than a single treatment.
- To maintain a reduction in speed downstream of the transition zone, additional treatments should be provided within the rural town; otherwise, speeds may increase within the rural town.

- Appropriate use of landscaping elements such as grass, shrubs, and trees which change in composition and degree of formality along the length of the transition zone can reinforce the changing characteristics of the environments.
- Consideration should be given to prohibiting passing within the transition zone.

Transition zone treatments may include:

- Geometric design changes such as median islands, roundabouts, and roadway narrowing;
- Traffic control devices such as transverse pavement markings and speed-activated feedback signs;
- Roadside features such as welcome signs and gateway landscaping; and
- Surface treatments such as transverse rumble strips and colored pavement.

Additional details concerning design of transition zones can be found in *Design Guidance for High-Speed to Low-Speed Transition Zones for Rural Highways* (*31*).

7.2.20 Design of Arterials in the Rural Town Context

As noted in Section 7.2.2, design speeds of 20 to 45 mph [30 to 70 km/h] are generally appropriate for arterials in the rural town context. Design speeds and posted speed limits may be decreased in stages as drivers leave the rural environment and approach the center of a rural town. On-street parking is seldom needed on arterials in the rural context, but may be vital to the economic success of businesses in the central portion of a rural town. On-street parking may also help in creating an appropriate low-speed environment within the rural town. Pedestrian and bicyclist flows may increase within rural towns creating a need for pedestrian and bicycle facilities. Rural towns may differ in their appropriate speed environment and needs for parking, pedestrian, and bicycle facilities, just as the suburban, urban, and urban core contexts in urban areas differ. Flexibility in the development of design features is appropriate to meet these varying needs in rural towns. Alternative design approaches and further guidance may be found in the discussion of arterials in urban areas in Section 7.3 and in two relevant publications that address the rural town context: *When Main Street is a State Highway* (*23*) developed by the Maryland Department of Transportation and *Main Street… When a Highway Runs Through It* (*24*), developed by the Oregon Department of Transportation. Further guidance may be found in FHWA's *Small Town and Rural Multimodal Networks Guide* (*11*).

7.3 ARTERIALS IN URBAN AREAS

This section presents guidance on the design of arterial streets in urban areas. Arterials in urban areas are designed with a flexible approach to meet the needs of the suburban, urban, and urban core contexts. As an arterial street moves from the suburban context to the urban context, and then to the urban core context, the emphasis on maintaining higher vehicle operating speeds decreases, the importance of providing on-street parking in appropriate locations increases, and the pedestrian, bicycle, and transit flows that need to be served, will likely increase. A flexible and balanced design approach to serve all transportation modes appropriately given the context and community values should be applied. The balance among transportation modes may differ between projects based on the demand flows for each transportation mode, community goals and values, and established areawide and corridor plans. The design guidance given below should be adapted to the context and needs of each individual facility and corridor.

7.3.1 General Characteristics

Arterials in urban areas carry large or moderate traffic volumes within and through urban core, urban, and suburban contexts. Their design varies from freeways and expressways with fully controlled access to two-lane streets, although grade-separated facilities are addressed in Chapter 8. The type of arterial selected is closely related to the level and quality of service desired for all users and to the context in which it is located.

A principal objective for an arterial in an urban area should be mobility of all users in an appropriate balance for the context of the facility and the appropriate degree of service to local development. Where full restriction of local access is not practical or preferred, designs that incorporate modern access management principles are desirable. Such designs could include roadways that provide separate turn lanes, pedestrian facilities, bicycle lanes, transit lanes and stops, consolidated driveways, medians, parking bays, or one-way streets. Most arterials in an urban area provide some access to abutting property. Such access service should, however, not unduly hinder the arterial's primary function of serving traffic and other user movements along and across the facility.

Before designing or redesigning an arterial in an urban area, it is important to establish the extent and need for such a facility from the perspective of all legal users of the facility. Once the need is established, steps should then be taken to protect the ability of the arterial to serve all users at the desired level and quality of service from future changes, such as strip development or the unplanned location of a major traffic generator. Development along an arterial in an urban area should be anticipated regardless of the urban area size. However, with proper planning and design, such development may be properly coordinated with the purpose and goals of that portion of the arterial network, including serving through travel, as appropriate. A well-designed arterial can complement such development and meet the needs of all users.

Arterials in urban areas are functionally divided into two classes, principal and minor. These classes are discussed in detail in Chapter 1. The system of arterials in urban areas, which includes arterial streets and freeways, normally serves the major activity centers of a metropolitan area, the highest traffic volume corridors, and the longest trips. The portion of the system of arterials in urban areas, either planned or existing, on which access is not fully controlled is addressed in this section of the chapter. Design of freeways is addressed in Chapter 8.

7.3.2 General Design Considerations

In the development of a transportation improvement program, routes selected for improvement as arterials may comprise portions of an existing street system, or they may be anticipated locations on new alignments through relatively undeveloped areas or suburban contexts. Usually, they will be existing streets because, historically, the need for improving existing streets has surpassed the availability of resources. As a consequence, street improvements tend to lag, rather than lead, land-use development.

Major improvement of existing arterials can be extremely costly, particularly where multiple utilities require relocation and additional rights-of-way need to be acquired through highly developed areas. Accordingly, it is often appropriate to apply flexibility in selection of design elements, controls, and criteria that are below the values used where sufficient right-of-way is available or can be acquired economically.

7.3.2.1 Design Speed

Design speeds for arterials in urban areas vary greatly between the three urban area contexts—suburban context, urban context, and urban core context. Design speeds for arterials in the suburban context generally range from 30 to 55 mph [50 to 90 km/h]. Design speeds for arterials in the urban context are generally lower and typically range from 25 to 45 mph [40 to 70 km/h]. Design speeds for arterials in the urban core context are generally 30 mph [50 km/h] or less. Design speed should be selected as described in Section 2.3.6.

7.3.2.2 Design Traffic Volumes

The design of arterials in urban areas should be based on motorized and nonmotorized traffic and other user data developed for the design year, normally 20 years into the future. The design hourly volume (DHV) is the most reliable traffic volume measure representing the vehicular traffic demand for use in design of arterials in urban areas. While future estimates of transit and nonmotorized use may not be available from traditional sources, the designer may use available planning and land use documents to assist in determining future levels of nonmotorized demand. Sometimes capacity analysis, which is used to determine whether a particular design can provide a desired level of service for conditions represented by the design traffic volume, is also used as a design tool. The HCM (*35*) includes capacity analysis approaches for all roadway users and should be consulted when multiple user modes exist or are expected along a facility. Refer to Sections 2.3 and 2.4 for further information on design traffic volumes and capacity analysis.

Design volumes for nonmotorized users should also be developed for the design of facilities in urban areas. Several guidelines are available that address forecasting pedestrian and bicycle volumes for urban arterial design projects (*25, 26, 27*).

7.3.2.3 Level of Service

When designing for future design year, arterials in urban areas and their auxiliary vehicle facilities (e.g., turning lanes, intersections, interchanges, and traffic control signals and systems) can be designed for level of service C or D. The choice of the design level and quality of service for a facility involves striking an appropriate balance between the needs of and service levels for motor vehicles, pedestrians, transit, and bicycles; the context, the community; and the degree of confidence in future land use development and trip generation projections. In heavily developed sections of metropolitan areas, the use of level of service D may be appropriate, although it may be impractical to achieve even this level of service in constrained settings. In rapidly developing urban areas, at least providing adequate right-of-way and appropriate drainage and grading for a Level of Service C for all users should be considered. While motor-vehicle level of service is calculated in a quantitative manner using numerical formulas, quality of service for pedestrians and bicycles is often a more qualitative analysis and may be a more appropriate process for analyzing facility performance, including accessibility, potential conflicts with motor vehicles, stress, and overall acceptable accommodation. For additional guidance on determining the level of service for all modes for a specific facility, refer to Sections 2.3 and 2.4, the HCM (*35*), and the FHWA *Guidebook for Developing Pedestrian and Bicycle Performance Measures* (*14*).

7.3.2.4 Sight Distance

Providing adequate sight distance is important in the design of arterials in urban areas. Sight distance affects normal operational characteristics, particularly where roadways carry high traffic volumes, and are important to the visibility of pedestrians and bicyclists as well. The sight distance values given in Table 7-1 are also applicable to the design of arterials in urban areas. Design values for intersection sight distance are presented in Section 9.5.

7.3.2.5 Alignment

The alignment of an arterial in an urban area should be developed in accordance with its design speed, desired operating speed, and context, particularly where a principal arterial is to be constructed on a new location and is not restricted by right-of-way constraints. There are many situations, however, where this is not practical. An example of this is the need to shift (deflect) the alignment of through lanes to accommodate left-turn lanes and other design features in an intersection area. Under such circumstances, the intersection alignment should be consistent with the guidance in Section 9.4. It is desirable to use the best alignment design practical since curves on arterials in urban areas are often not superelevated in the low-speed range (see discussion on superelevation in Section 7.3.2.7 for further explanation).

7.3.2.6 Grades

The grades selected for an arterial in an urban area may have a significant effect on its motor-vehicle operational performance and can also effect pedestrian and bicycle operations. For example, steep grades affect truck speeds and stopping distances, as well as the overall capacity on the facility for all user modes. On arterials in urban areas having large numbers of trucks and operating near capacity, flatter grades should be considered to avoid undesirable speed reductions. Bicyclists will slow substantially on uphill grades, making provision of dedicated bicycle facilities desirable. Steep grades may also result in operational problems at intersections, particularly during adverse weather conditions, and may adversely affect the ability to provide accessible adjacent pedestrian facilities. For these reasons, it is desirable to provide the flattest grades practical while providing 0.3 percent minimum (0.5 percent desirable) gradients to provide adequate longitudinal drainage in curbed sections. The recommended maximum grades for arterials in urban areas are presented in Table 7-4. Where steep grades cannot be flattened, climbing lanes may be considered based on the warrants presented in Section 3.4.3.

Table 7-4a. Maximum Grades for Arterials in Urban Areas, U.S. Customary

Type of Terrain	Maximum Grade (%) for Specified Design Speed (mph)								
	20	25	30	35	40	45	50	55	60
Level	8	7	7	7	7	6	6	5	5
Rolling	10	10	9	8	8	7	7	6	6
Mountainous	13	12	11	10	10	9	9	8	8

Table 7-4b. Maximum Grades for Arterials in Urban Areas, Metric

Type of Terrain	Maximum Grade (%) for Specified Design Speed (km/h)							
	30	40	50	60	70	80	90	100
Level	8	7	7	7	6	6	5	5
Rolling	10	10	9	8	7	7	6	6
Mountainous	11	12	11	10	9	9	8	8

7.3.2.7 Superelevation

Curves on low-speed, curbed arterial streets are usually not superelevated. Difficulties associated with drainage, ice formation, driveways, pedestrian crossings, bicycle accommodation, and the effect on adjacent developed property should be evaluated when superelevation is considered. Section 3.3 on "Horizontal Alignment" provides a more detailed discussion of superelevation. When little or no superelevation is provided on curves for low-speed arterial streets, the Method 2 distribution of superelevation discussed in Section 3.3.6 is usually used. Supplemental guidance applicable to arterials in both rural and urban areas is presented in the discussion of superelevated cross sections presented in the earlier discussion of arterials in rural areas in Section 7.2.11.9.

7.3.2.8 Cross Slope

Sufficient cross slope for adequate pavement drainage is important on arterials in urban areas. The typical problems related to splashing and hydroplaning are compounded by heavy traffic volumes and curbed sections, especially for higher speeds. Cross slopes should range from 1.5

to 3 percent; the lower portion of this range is appropriate where drainage flow is across a single lane and higher values are appropriate where flow is across several lanes. Even higher cross-slope rates may be used for parking lanes; however, accessible parking spaces should have the flattest slope possible. The overall cross section should provide a smooth appearance without sharp breaks, especially within pedestrian access routes where specific accessibility guidelines apply (*36*, *37*, *38*). Because arterials in urban areas are often curbed, it is necessary to provide for longitudinal as well as cross-slope drainage. The use of higher cross-slope rates also reduces flow on the roadway and ponding of water due to pavement irregularities and rutting. Section 4.2.2, "Cross Slopes" provides additional guidance.

7.3.3 Cross-Sectional Elements

7.3.3.1 Roadway Widths

The roadway width should be adequate to accommodate through-travel lanes and turn lanes, on-street parking and/or shoulders if provided, bicycle accommodation where appropriate, medians, curbs, and appropriate clearances from curb or barrier face. When parking lanes are provided, consideration may be given to providing a width adequate to allow peak hour or future operation as a travel lane. When future context changes are anticipated along a corridor, consideration may be given to converting through lanes to transit, bicycle, or parking lanes. In many instances at intersections, the parking lane is used to provide a right-turn lane or used as a through-travel lane in order to provide additional width for a left-turn lane.

7.3.3.2 Lane Widths

Lane widths on through-travel lanes may vary from 10 to 12 ft [3.0 to 3.6 m]. Lane widths of 10 ft [3.0 m] may be used in more constrained areas where truck and bus volumes are relatively low and speeds are less than 35 mph [60 km/h]. Lane widths of 11 ft [3.3 m] are used quite extensively for urban arterial street designs. The 12-ft [3.6-m] lane widths are desirable, where practical, on high-speed, free-flowing, principal arterials.

Under interrupted-flow operating conditions at low speeds (45 mph [70 km/h] or less), narrower lane widths are normally adequate and have some advantages. For example, reduced lane widths allow more lanes to be provided in areas with restrictive right-of-way, may help in reducing operating speeds, and allow shorter pedestrian crossing times because of reduced crossing distances. Arterials with reduced lane widths are also more economical to construct and produce less stormwater runoff. An 11-ft [3.3-m] lane width is often adequate for through lanes, continuous two-way left-turn lanes, and lanes adjacent to a painted median. Left-turn and combination lanes used for parking during off-peak hours and for traffic during peak hours may be 10 ft [3.0 m] in width. If provision for bicyclists is to be made, see the AASHTO *Guide for the Development of Bicycle Facilities* (*7*).

If substantial truck or bus traffic is anticipated, additional lane width may be desirable. The widths needed for all lanes and intersection design controls should be evaluated collectively with consideration of all user modes and the adjacent land use. For instance, a wider right-hand lane that provides for right turns without encroachment on adjacent lanes may be attained by providing a narrower left-turn lane. Local practice and experience regarding lane widths should also be evaluated.

7.3.3.3 Curbs and Shoulders

Shoulders may be desirable on high-speed (50 to 60 mph [80 to 100 km/h]) arterials in urban areas. They provide additional maneuvering room and space for immobilized vehicles and/or bicycles. They may also serve as speed-change lanes for vehicles turning into driveways and intersections and provide storage space for plowed snow.

The use of shoulders on arterial streets is generally limited by restricted right-of-way and the need to use the available right-of-way for travel lanes, parking lanes, transit lanes, bicycle lanes, pedestrian facilities, and other needs. Where the abutting property is used for commercial purposes or consists of high-density residential development, a shoulder, if provided, is subject to such heavy use in serving local traffic that the pavement strength of the shoulder should be about the same as that for the travel lanes. In urban and suburban areas, the outside edges of shoulders are often curbed and a closed drainage system provided to minimize the amount of right-of-way needed. In addition, curbs are often appropriate in heavily developed areas as a means of controlling access.

In those instances where sufficient right-of-way exists to consider shoulders on high-speed arterials, refer to the discussion on shoulders on arterials in rural areas in Section 7.2.3 for guidance. Where providing shoulders is not desired or practical, and curbs are to be used, refer to Section 4.7.3, "Curb Placement."

7.3.3.4 Number of Lanes

The number of lanes varies, depending on traffic demand, presence and needs of other users, and availability of the right-of-way, but the typical range for arterials in urban areas is four to eight through lanes in both directions of travel combined. Many minor arterials may have two through-travel lanes, one in each direction. A capacity analysis for all users should be performed to determine the proper number of lanes in consideration of the space needed to accommodate all users of the right-of-way. In addition, roadways are sometimes widened through intersections by the addition of one or two auxiliary lanes to accommodate turning vehicles. Section 2.4 presents additional information on capacity analysis.

7.3.3.5 Medians

Medians are a desirable feature of arterials in urban areas and should be provided where space permits. Medians and median barriers are discussed in Sections 4.10.2 and 4.11. In urban areas, where right-of-way is often limited, it is frequently necessary to determine how best to allocate the available space between border areas, traveled way, and medians. On lower-volume, lower-speed arterials in urban areas, the decision is often resolved in favor of no median at all. However, a median 4 ft [1.2 m] wide is normally better than none for some contexts, and it should be noted that any additional median width may reduce crash severity for vehicles that run off the road and can improve operation between intersections. Medians provide space for landscaping and other enhancement, and can also be a benefit to pedestrians by providing a refuge area, allowing pedestrians to cross one direction of traffic at a time, provided that the median is at least 6 ft [1.9 m] wide and are accessible to and usable by individuals with disabilities.

At intersections in urban and suburban contexts, median widths should be limited, whenever practical, to those widths needed to accommodate pedestrian refuge and appropriate left-turn treatments for current and future traffic volumes. Pedestrian refuge has been shown to reduce crash frequency and is often preferable where arterials have four or more lanes. At intersections where left turns are made, a left-turn lane is desirable to increase capacity and reduce crash frequencies. To accommodate left-turn movements, the median should be at least 10 ft [3.0 m] wide. A minimum 4-ft [1.2-m] medial separator between the turning lane and the opposing traffic lane is desirable. With wider medians, consideration should be given to off-setting the left-turn lanes to provide maximum visibility between left-turning vehicles and opposing traffic flows. Refer to Sections 9.7 and 9.9 for additional guidance concerning provision of dual left-turn lanes and other special intersection treatments.

The median configurations shown in Figures 7-4B, 7-4C, 7-4D, and 7-4E are appropriate for suburban, urban, or urban core settings. The type of median treatment used is usually dependent on context, pedestrian crossing volumes, local practice, and available right-of-way widths. The median type selected should be compatible with the needs of drainage and street hardware.

Median openings on high-speed divided arterials with depressed or raised curbed medians should be carefully considered. Such openings should only be provided for street intersections, for U-turns, or for major developments. Spacing between median openings should be adequate to allow for introduction of left-turn lanes and anticipated storage needs of left-turn queues.

On higher-speed arterials where intersections are relatively infrequent (e.g., 0.5 mi [1.0 km] or more apart) and there is no existing or expected pedestrian crossing needs, the median width may be varied by using a narrow width between intersections for economy and then gradually widening the median on the intersection approaches to accommodate left-turn lanes. This solution is rarely practical, however, and should generally not be used where intersections are closely spaced because the curved alignment of the lane lines may result in excessive maneuvering by drivers to stay within the through lanes. It is far more desirable that the median be of uniform width. Where a narrow median is provided on a high-speed facility, consideration should be given to inclusion of a median barrier. Refer to the AASHTO *Roadside Design Guide* (*6*) for guidance on use and placement of median barriers.

For a street with an odd number of lanes, typically three or five, the center lane is often used to provide a deceleration and storage lane for left-turning vehicles. Left-turn bays are typically marked in advance of intersections. The center lane between left-turn bays is typically used for vehicles making midblock left turns. In some cases, the center lane is designated for "Left-Turn Only" from either direction, commonly referred to as two-way left-turn lane (TWLTL) design, without specially marked bays at minor intersections. This type of operation works well where the speed on the arterial highway is relatively low (25 to 45 mph [40 to 70 km/h]) and there are no heavy concentrations of left-turning traffic. Additional guidance is available in NCHRP Report 780, *Design Guidance for Intersection Auxiliary Lanes* (*16*).

Where an arterial in an urban area passes through a developed area having numerous cross streets and commercial or residential driveways, and where it is impractical to limit left turns, the two-way left-turn lane is often the best solution. Because left-turning vehicles are provided a separate space to slow and wait for gaps in traffic, the interference to traffic in through lanes is minimized. Continuous two-way left-turn lanes should be identified by lane and arrow markings placed in accordance with the MUTCD (*12*). Figure 7-8 shows an example of a two-way left-turn lane. For further information, see Section 4.11, "Medians," and Section 9.11.7, "Midblock Left Turns on Streets with Flush Medians."

A raised curbed median is typically used on low-speed (45 mph [70 km/h] and below) arterials in urban areas. This median type is used where it is consistent with the context (urban core, urban, or suburban), and where it is desirable to manage access and stormwater along an arterial street and provide delineation between motorized and nonmotorized users. Raised curbed medians provide a refuge for pedestrians and a good location for landscaping, signs, and other appurtenances. In addition, in snow-belt areas, raised curbed medians provide positive delineation and can provide space for plowed snow.

However, raised curbed medians also present disadvantages that should be considered. On arterials in urban areas serving high-speed (50 mph [80 km/h] and above) traffic, a raised curbed median does not normally prevent pedestrian or cross-median crashes unless a median barrier is also provided. If accidentally struck, the raised curb may cause drivers to lose control of their vehicles. Also, such medians can be difficult to see at night without appropriate fixed-source lighting or proper delineation. In some cases, the prevention of midblock left turns may cause operational problems at intersections because of increased concentrations of left-turning or U-turning traffic.

Source: Charlotte Department of Transportation

Figure 7-8. Continuous Two-Way Left-Turn Lane

Median crossings should be accessible for persons with disabilities at all legal crosswalks, whether marked or unmarked.

The foregoing traffic operational disadvantages of raised curbed medians can be largely eliminated by use of flush medians or low-profile sloped curbs. However, flush medians are difficult to see under wet nighttime conditions, may become indiscernible under the lightest of snowfall conditions, and provide little refuge for pedestrian crossings. Visibility of flush medians can be improved by use of a contrasting pavement texture and by improved delineation, such as the use of reflectorized pavement markers. The use of raised bars or blocks has proven to be an ineffective median treatment and should be avoided.

When a two-lane arterial in a suburban context is proposed for improvement to a multilane facility with a median, access management principles suggest that a raised curbed median is more desirable than a flush median. The limiting of left-turns except at intersections discourages uncontrolled development and access to the highway and promotes improved traffic operations.

Special consideration should be given to the median width where intersections are provided. Research in NCHRP Report 375 (*21*) found that most types of undesirable driving behavior in the median area of divided highway intersections are associated with competition for space by vehicles traveling through the median in the same direction. The potential for such problems is generally greater at urban and suburban rather than at rural intersections, where volumes of turning and crossing traffic are lower. Types of undesirable driving behavior observed include side-by-side queuing, angle stopping, and encroaching on through lanes of a divided highway. At urban and suburban unsignalized intersections, the frequency of crashes and undesirable driving behavior were observed to increase as the median width increased. Thus, medians at urban and suburban unsignalized intersections should not be wider than necessary considering the needs of other user modes.

Urban and suburban unsignalized intersections with median widths from 30 to 50 ft [9 to 15 m] appear to operate quite well, although they may experience slightly higher crash rates than intersections with narrower medians. However, urban and suburban intersections with medians wider than 50 ft [15 m] have more crashes, and intersections with medians wider than 60 ft [18 m] are difficult to signalize properly (*21*).

Median widths at urban and suburban signalized intersections should be determined primarily by the space needed in the median for current or future left-turn treatments, and should not be wider than necessary (*21*). Median widths of more than 60 ft [18 m] are undesirable at intersections that are signalized or that may need signalization in the foreseeable future. The efficiency of signal operations decreases as the median width increases, because drivers need more time to traverse the median and special detectors may be needed to avoid trapping drivers in the median at the end of the green phase for traffic movements passing through the median. Furthermore, if the median becomes so wide that separate signals are needed on the two roadways of the divided highway, delays to motorists will increase substantially. However, careful attention should be given to vehicle storage needs in the median area between the two signals. At locations with substantial crossing and turning volumes of larger vehicles, such as school buses or trucks, it may be appropriate to provide enough width to store such vehicles in the median without encroaching on the through lanes of the major road.

Uncurbed, unpaved narrow medians often present problems for turning movements at intersections because vehicles tend to run off the roadway edges. To minimize this problem, the provision of edge lines and sufficient paved area beyond the edge lines provides positive guidance and will accommodate the turning paths of passenger cars and occasional large vehicles.

A median barrier may be desirable on some arterial streets with higher speed traffic. A barrier provides a positive separation of traffic and discourages indiscriminate pedestrian crossings. Where the median barrier is terminated at cross streets and other median openings, it should have a crashworthy terminal or terminal end appropriate for the speed of traffic. Further discussion on treatment of the ends of barriers is presented in the *Roadside Design Guide* (*6*). Additional information on median barriers and median treatments at intersection areas is found in Sections 4.10.2 and 9.8, respectively. The information on medians and median barriers in

Sections 4.10.2 and 4.11 is especially pertinent to arterials in urban areas since they need the most varied application of these features.

7.3.3.6 Drainage

An adequate drainage system to accommodate design runoff should be included in the design of every arterial street. Inlets that are bicycle-compatible should be located adjacent to and upstream of intersections and at intermediate locations where needed. Where a shoulder or parking lane is provided, the full width of the shoulder or parking lane may be utilized to conduct surface water to the drainage inlets. Where no shoulder or parking lane is provided, one-half of the outside traffic lane and curb offset may be utilized to conduct surface drainage, provided two or more traffic lanes exist in each direction. Ponding of water at low points in the traveled way on arterial streets is undesirable. The width of water spread on the roadway should not be substantially greater than the width of spread encountered on continuous grades. Highways with design speeds greater than 45 mph [70 km/h] will have a higher potential for hydroplaning than highways with lower speeds when the traveled way is covered with water. Additional inlets should be provided in sag locations to avoid ponding of water where the grade flattens to zero percent and to mitigate flooding should an inlet become clogged. Chapter 4 has comprehensive discussions concerning drainage.

7.3.3.7 Parking Lanes

Where parking is needed to contribute to an urban context or where adequate off-street parking facilities are not available or practical, parallel or angle parking may be considered on lower-speed arterials as long as the capacity provided by the through lanes for motor vehicles and bicycles is considered. However, parking is highly undesirable on higher-speed roadways (50 mph [80 km/h] and above) and generally not used on facilities in the 40- to 45-mph [60- to 70-km/h] range.

Passenger vehicles parked adjacent to a curb will occupy, on the average, approximately 7 ft [2.1 m] of street width. Therefore, the total parking lane width for passenger cars should be 7 to 10 ft [2.1 to 3.0 m]. This width is also adequate for an occasional parked commercial vehicle. To accommodate usage by bicyclists, as well as passenger cars, a combined width of 12 to 14 ft [3.6 to 4.2 m] is desirable, and could be wider if a buffer is provided from the through lanes or parked vehicles or both. Refer to the AASHTO *Guide for the Development of Bicycle Facilities* (*7*) and the FHWA guide on *Achieving Multimodal Networks: Applying Design Flexibility and Reducing Conflicts* (*15*). Where it is unlikely that there will be a future need to use the parking lane as a through lane, a parking lane width as narrow as 7 ft [2.1 m] may be acceptable. On curbed roadways, the width of the parking lane is measured to the face of curb. Where on-street parking is provided, a portion of that parking should be accessible for use by persons with disabilities. Additional width may be needed to provide an access aisle, so accessible parking needs should be assessed early in project design. Further guidance may be found in the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*36*).

A parking lane less than 11 ft [3.3 m] in width measured to the face of curb is usually considered undesirable if future use of the parking lane for through traffic is anticipated. Such a lane can be used as an additional through-traffic lane during peak hours by prohibiting parking during these hours. A parking lane 10 ft [3.0 m] in width is typically acceptable for use as a storage lane for turning vehicles at signalized intersections by prohibiting parking for some distance upstream from the intersection.

The marking of parking spaces on arterial streets encourages more orderly and efficient use where parking turnover is substantial and it also tends to prevent encroachment on fire hydrant zones, bus stops, loading zones, approaches to corners, clearance space for islands, and other zones where parking is prohibited. Typical parking-space markings are shown in the MUTCD (*12*).

7.3.3.8 Borders and Sidewalks

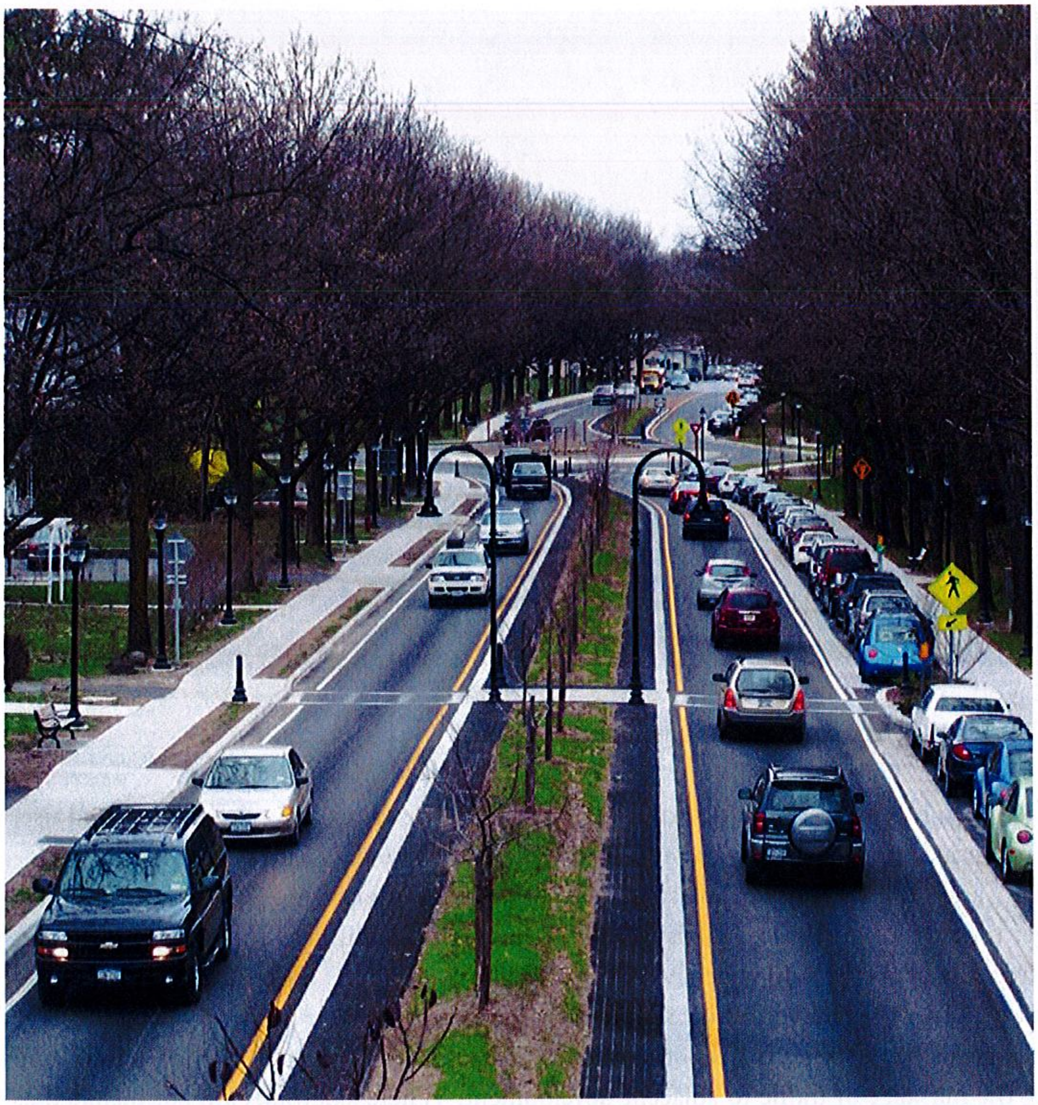
The border is the area between the roadway and the right-of-way line that separates traffic from adjacent homes and businesses. For a minimum section in a residential area, or any contexts where pedestrians are present or expected in the future, the border area should include a sidewalk and a buffer strip between the sidewalk and curb. Transit stops and multi-use paths for pedestrians and bicycles may also be placed in the border area. Figure 7-9 illustrates an arterial street in a residential area and shows curbs, a parking lane, curb cuts for driveways, and sidewalks. This type of arterial features a turf buffer strip that is provided between the sidewalk and the curb. In addition, vertical-curb and gutter sections are employed on the outside of parking lanes that may also serve as shoulders. In blocks that are fully developed with retail stores and offices, the entire border area is usually devoted to a wider sidewalk that also provides space for light poles, planters, trees, parking and traffic signs, parking meters, fire hydrants, mail boxes, and other types of street furniture.

Source: City of Charleston, WV

Figure 7-9. Arterial Street in a Residential Area

Some factors to be considered in determining border widths are existing and future land use, vehicle operating speed, existing and future volumes of all modes, width of sidewalk for retail activity and pedestrian needs, off-street bicycle facilities, transit stops, snow storage, storm drainage, traffic control devices, roadside appurtenances, and utilities. The minimum border should typically be 8 ft [2.4 m] wide and often 12 ft [3.6 m] or more, depending on context and nonmotorized user needs. Every effort should be made to provide wide borders not only to serve functional needs but also as a matter of aesthetics, reducing crash frequencies for all users, and reducing the nuisance of traffic to adjacent development. Where sidewalks are not included as a part of the initial construction, the border should be sufficiently wide to provide for their future installation. The border can often be graded for future sidewalk installation, and driveways constructed to provide accessible crossings, during initial construction at little additional cost, making the installation of sidewalk in the future less disruptive to adjacent businesses. For further information, see Section 7.3.9, "Bicycle and Pedestrian Facilities." Where bicycle traffic is anticipated or is to be served on arterial streets, provisions to accommodate bicycles should be in accordance with the AASHTO *Guide for the Development of Bicycle Facilities* (7).

Figure 7-10 illustrates a divided arterial street with a parking lane in a residential area.



Source: New York State DOT

Figure 7-10. Divided Arterial Street with Parking Lane

7.3.3.9 Right-of-Way Width

The width of right-of-way for the complete development of an arterial street is influenced by both vehicular and nonmotorized traffic demands, topography, land use, cost, intersection design, and the extent of ultimate expansion. The width of right-of-way should be the summation of the various cross-sectional elements (existing and planned): through-traveled ways, parking lanes, bicycle lanes, medians, auxiliary lanes, shoulders, borders, sidewalks, and, where appropriate, frontage roads, roadside clear zones, sideslopes, drainage facilities, utility appurtenances, and retaining walls. The width of right-of-way should be based on the dimensions of each element that best accommodate in consideration of desired operating conditions for existing and future contexts. The designer is confronted with the problem of providing an overall cross section that will appropriately serve all modes within a limited width of right-of-way. Right-of-way widths in urban areas are governed primarily by community goals and context plans, economic considerations, physical obstructions, or environmental concerns. Along any arterial route, conditions of development and terrain vary, and accordingly, the availability of right-of-way varies. For this reason, the right-of-way on a given facility should not be a fixed width predetermined on the basis of the most critical point along the facility. Instead, a desirable right-of-way width should be provided along most, if not all, of the facility.

7.3.4 Roadside Design

There are several primary considerations for roadside design along the traveled way for arterials in urban areas. From a motor-vehicle standpoint, clear zones and lateral offset are key design considerations, particularly in higher-speed arterial settings. As design and operating speeds decrease, other considerations become as important, or possibly more important, in order to provide a balanced roadway design for all users. These considerations may include providing access and mobility for nonmotorized modes, facilitating transit operations, enhancing aesthetics, supporting the local economy, and achieving other community goals.

7.3.4.1 Clear Zones

While the values provided in the AASHTO *Roadside Design Guide* (*6*) are appropriate for freeways and other controlled-access facilities, in an urban environment the right-of-way is often limited and, in most cases, it is not practical to establish a clear zone using the guidance in the AASHTO *Roadside Design Guide*. Urban environments are often characterized by sidewalks beginning at the face of curb, enclosed drainage, numerous fixed objects (signs, utility poles, luminaire supports, fire hydrants, sidewalk furniture, etc.), adjacent retail activity, and frequent traffic stops. These environments typically have lower operating speeds, with frequent signalized intersections, and in many instances on-street parking is provided.

On curbed facilities located in transition areas between rural and urban settings, there may be opportunity to provide greater lateral offset in the location of fixed objects. These facilities are generally characterized by higher operating speeds and have sidewalks separated from the curb by a grass strip. Although establishing a clear zone commensurate with the suggested values in the *Roadside Design Guide* (*6*) may not be practical due to right-of-way constraints, consideration should be given to establishing a reduced clear zone, or incorporating as many clear zone concepts as practical, such as removing roadside objects or making them crashworthy. The location of fixed objects should also be closely coordinated with any existing or planned pedestrian and bicycle facilities in the border areas, paying particular attention to the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*36*).

7.3.4.2 Lateral Offset

On arterials in the urban context or urban core context, a lateral offset to vertical obstructions (e.g., signs, utility poles, luminaire supports, and fire hydrants, and including breakaway devices) is needed to accommodate motorists operating on the highway. The lateral offset to obstructions helps to:

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

Lateral offset is defined in Section 4.6.2. Further discussion and suggested guidance on the application of lateral offsets is provided in the AASHTO *Roadside Design Guide* (6).

Where a curb is used, the lateral offset is measured from the face of the curb. A minimum of 1.5 ft [0.5 m] should be provided from the face of the curb, with 3 ft [1.0 m] at intersections to accommodate turning trucks and improve sight distance. Consideration may be given to providing more than the minimum lateral offset to obstructions where practical by placing fixed objects behind the sidewalk. Traffic barriers, where needed, should be located in accordance with the AASHTO *Roadside Design Guide* (6), which may recommend that the barrier should be placed in front of or at the face of the curb.

On facilities with shoulder width less than 4 ft [1.2 m] and without curb, a minimum lateral offset of 4 ft [1.2 m] from the edge of the traveled way should be provided. As noted above, the location of fixed objects should also be closely coordinated with any existing or planned pedestrian, bicycle, and transit facilities in the border areas, paying particular attention to the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (36).

7.3.5 Structures

7.3.5.1 New and Reconstructed Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the current *AASHTO LRFD Bridge Design Specifications* (9). The design loading should be the HL-93 calibrated live load designation.

The minimum clear width for new bridges on arterial streets should be the same as the curb-to-curb width of the street including any existing or proposed off-street bicycle paths and on-street bicycle lanes. In addition, on streets with sidewalks, the sidewalks should also continue across the bridge. Adequate separation from motor-vehicle traffic should be provided to adjacent nonmotorized facilities. Due to the long life expected from structures, providing sidewalks on bridges may eliminate the need for widening in the future as development occurs. On long bridges, defined as bridges with overall lengths in excess of 200 ft [60 m], the shoulders may be reduced to 4 ft [1.2 m] where shoulders or parking lanes are provided on the arterial. For further relevant discussion, see Sections 4.7, "Curbs;" 4.10, "Traffic Barriers;" and 4.17.1, "Sidewalks."

7.3.5.2 Vertical Clearances

New or reconstructed structures should provide 16-ft [4.9-m] vertical clearance over the entire roadway width. Existing structures that provide clearance of 14 ft [4.3 m], if allowed by local statute, may be retained. In highly urbanized areas, a minimum clearance of 14 ft [4.3 m] may be provided if there is an alternate route with 16-ft [4.9-m] clearance. Consideration should be given to providing additional clearance for future resurfacing of the underpassing road. The vertical clearance to sign supports and to bicycle and pedestrian overpasses should be 1.0 ft [0.3 m] greater than the highway structure clearance.

7.3.6 Traffic Barriers

Traffic barriers are sometimes used in restricted areas, at separations, and in medians of arterials in urban areas. The barrier should be compatible with context and the desired visual quality and should be installed in accordance with accepted practice. Exposed ends should be treated with crashworthy designs or other appropriate means. For further information, refer to the AASHTO *Roadside Design Guide* (*6*).

7.3.7 Railroad–Highway Grade Crossings

Railroad–highway crossings on an arterial in an urban area can often be the most disruptive feature affecting its operation. Crossings that are frequently occupied or occupied during high-volume traffic periods should be treated by providing a grade separation. Crossings that are occupied only infrequently or during off-peak traffic periods may be operated as an at-grade crossing with high-type traffic control, such as gate-equipped automatic flashing signals.

At-grade crossings that involve pedestrian sidewalks or bicycle routes that are not perpendicular to the railroad may need additional sidewalk width or paved shoulder width to allow pedestrians and bicyclists to maneuver over the crossing. For further information, see the AASHTO *Guide for the Development of Bicycle Facilities* (*7*) and the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*3*).

7.3.8 Access Management

7.3.8.1 General Features

Partial control of access and the application of access management techniques are desirable on an arterial in an urban area. Effective access management will not only enhance the initial motor-vehicle level of service of a facility but may also preserve that original level of service as further development occurs. While access to abutting property is usually required, it should be carefully regulated to limit the number of access points and their locations. Access management is especially important on intersection approaches on both the arterial and cross streets where auxiliary and storage lanes may be needed.

The location and design of driveways, together with parking and bicycle facilities, may make it difficult for motorists using driveways and approaching pedestrians and bicyclists to see one another. The application of various access management strategies at driveways has direct implications for reducing potential conflicts involving pedestrians and bicyclists at driveway locations. Any access management design effort should address all user modes that are affected by vehicle crossings. NCHRP Report 659, *Guide for the Geometric Design of Driveways* (*17*), provides design guidance for driveways on arterial and other streets including consideration of all users.

Access control and access management may be exercised by statute or through zoning ordinances, driveway regulations, turning and parking regulations, and effective geometric highway design. Implementation of any of these options should involve coordination with the community and adjacent property owners. For additional discussion on access control and access management, refer to Section 2.5.

7.3.8.2 Access Control by Statute

Where a high degree of access control is desired, it is usually accomplished by statute. When statutory control is applied to an arterial street, access is usually limited to the cross streets or to other major traffic generators.

7.3.8.3 Access Control by Zoning

Zoning can be used effectively to control the type of property development along an arterial and thereby influence the type and volume of traffic generated. In certain cases, it may be desirable to exclude land uses that generate heavy volumes of commercial traffic if, for various reasons, this class of vehicle cannot be accommodated readily by limitations in the highway geometrics.

Zoning regulations should require adequate off-street parking as a condition for approval of a building permit. Also, the internal arrangement of the land-use development should be such that the parking spaces are placed away from the street and with the building frontages closer to the sidewalk. This type of internal design minimizes congestion in the vicinity of the entrance at the street. Vehicles exiting from the parking facility to the arterial (or preferably to a cross street) should not impede traffic entering the parking facility from the arterial.

Subdivision or zoning ordinances should require that the developer of a major traffic generator provide a suitable connection to the arterial street (or preferably to a cross street) comparable to that for a well-designed street intersection serving a similar volume of traffic. If direct access to the arterial is provided, it should be understood that the intersection is subject to the same traffic control measures, including restrictions to turning movements, as are applicable elsewhere on the arterial. In suburban areas, developers may be required to provide an internal connection between adjacent properties or a rear connecting roadway for access to the properties to maintain a high traffic operational level of service and minimize the potential for crashes on the arterial.

7.3.8.4 Access Control through Driveway Regulations

Driveway controls can be effective in preserving the functional character of arterial streets. In heavily developed areas and areas with potential for intensive development, permits for driveways and entrances can be controlled to minimize interference with the free flow of traffic on the arterial and pedestrian accessibility along the sidewalk. Cooperatively consolidated and joint use of carefully located driveways is one method of providing property access while maintaining access control. In more sparsely developed areas with little potential for dense development, driveway controls are also desirable so that future driveways are located where there will continue to be minimum interference with the free movement of traffic.

7.3.8.5 Access Control through Geometric Design

Left turns in and out of local streets and adjacent properties can have a great effect on the operation of and the frequency of crashes on an arterial. Such movements can be prohibited by constructing a raised curbed median or by installing a median barrier. Left turns can be accommodated by U-turns at intersections, "jug-handle" configurations, or around-the-block movements. The effects of relocating midblock turns to these alternative locations should be carefully considered to evaluate this option. Additional information concerning the effects of midblock left-turn lanes can be found in NCHRP Report 395, *Capacity and Operational Effects of Midblock Left-Turn Lanes* (*10*). Right-turn-in and right-turnout arrangements are another important geometric design feature to control access to an arterial.

Frontage roads and grade separations provide the most effective access control. Fully developed frontage roads effectively control access to through lanes on an arterial street while providing access to adjoining property, separating local from through traffic, and permitting circulation of traffic along each side of the arterial. When used in conjunction with grade-separation structures at major cross streets, an arterial takes on many of the operating characteristics of a freeway.

Due to right-of-way restrictions, frontage roads are usually located immediately adjacent to the arterial. For this reason, careful attention should be given to proper signing to minimize the potential for wrong-way entry into the through lanes of the arterial. Efforts should be made to provide adequate storage distance for turning vehicles on the crossroad between the frontage road and the arterial, although this is often difficult because of restricted right-of-way width. If signalization is needed at the intersection of the crossroad and the frontage road, the operation of this signal should be coordinated with the signal at the intersection of the crossroad and the arterial.

General features of frontage roads and their design are discussed in Section 4.12. The effect of frontage roads on the design of intersections is addressed in Section 9.11.1, "Intersection Design Elements with Frontage Roads." Additional information concerning access management can be found in NCHRP Report 420, *Impacts of Access-Management Techniques* (*19*) and the TRB *Access Management Manual* (*34*).

7.3.9 Bicycle and Pedestrian Facilities

7.3.9.1 Bicycle Facilities

Bicycle usage can be expected on most arterials in urban areas and should be considered in arterial street design. In the absence of dedicated bicycle facilities, bicycle travel in the motor-vehicle travel lanes should be expected. Separate facilities, such as bike lanes, separated bike lanes, and shared-use paths help preserve capacity for motor vehicles while reducing potential conflicts with bicyclists. The AASHTO *Guide for the Development of Bicycle Facilities* (*7*) should be referenced for appropriate facility selection and design guidance.

Bicycle-vehicle conflicts occur at many locations including intersections, driveways, parallel through lanes, and other locations where bicyclists negotiate across moving lanes of motor-vehicle traffic. On lower classes of arterials with lower motor-vehicle volumes and speeds, these conflicts are important but less challenging to address than on higher-speed, higher-volume arterials. In those settings, the designer should carefully consider the conflict potential between motor vehicles and bicycles and incorporate design and operational elements that address these needs.

At signalized intersections, signal clearance times need to provide time for bicyclists to clear the intersection (see the AASHTO *Guide for the Development of Bicycle Facilities* (*7*)) and turn lanes on streets with bicycle lanes should follow the design guidance in the *Manual on Uniform Traffic Control Devices* (*12*).

7.3.9.2 Pedestrian Facilities

Most arterial streets need to accommodate both vehicles and pedestrians; therefore, the design should include sidewalks, crosswalks, and sometimes grade separations for pedestrians. Pedestrian facilities and control measures will vary, depending largely on the context, volume of pedestrian traffic, volume of vehicular traffic to be crossed, number of lanes to be crossed, number of vehicles turning at intersections, and location of transit stops.

As a general practice, sidewalks should be provided along arterial streets in urban areas, even though pedestrian traffic may be light. On some sections of arterial streets that traverse relatively undeveloped areas, no initial pedestrian demand may be present, and, therefore, sidewalks may not be needed initially. Because these areas will usually be developed in the future, the design should allow for the ultimate installation of sidewalks.

The major pedestrian–vehicular conflict usually occurs at roadway intersections and at midblock pedestrian crossings. On lower classes of arterials, especially at intersections with minor cross streets where turning movements are light, pedestrian facilities are usually limited to crosswalk markings. Features that help the pedestrian include fixed-source lighting, high-visibility pavement markings, refuge islands, traffic barriers, flashing beacons, and pedestrian signals. Such features are discussed in Chapter 4.

On higher-volume arterials (i.e., four to eight lanes wide with heavier traffic volumes), the conflicts between pedestrians and vehicles at intersections sometimes present a real challenge for designers. The challenge is especially acute where the arterial traverses a business district and there are intersections with higher volume cross streets. Although grade separations for pedestrians may be justified in some instances, crosswalks are the predominant form of crossing. Conflicts between pedestrians and vehicular traffic can be reduced by shortening pedestrian crossing distances by various means such as curb extension bulbs or narrower lanes, restricting left or right turns, providing median refuge, and separate pedestrian signal phases. The accommodation of pedestrians can have an effect on the capacity of intersections and should be evaluated during design.

On heavily-traveled arterials in and near developed areas, crosswalks may be provided at intersecting streets and at midblock locations, as appropriate. Enforcement of a ban on pedestrian crossings at an intersection is very difficult. A crossing should not be closed to pedestrians unless the benefits to traffic are sufficient to offset the inconvenience to pedestrians. In addition, indiscriminate closing of pedestrian crossings will lead to illegal crossing maneuvers. Therefore, proper and reasonable design for pedestrians is important.

Pedestrian walk signals should be provided at all signalized intersections where pedestrian facilities are present or planned. On exceptionally wide arterial streets, pedestrian signals may be mounted in the median as well as on the far side of the intersection and, where frontage roads exist, in the outer separators as well. Refer to the current MUTCD (*12*) and the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*3*) for additional information concerning installation and timing of pedestrian signals and the location of pedestrian actuation buttons.

Where intersections are channelized or a median is provided, consideration should be given to the use of curbing for those areas likely to be used by pedestrians for refuge when crossing the roadway. The curb offset should be consistent with the design criteria in Section 4.7.3.

The use of crosswalks at typical curbed-street intersections may be difficult for persons with disabilities. Curb ramps of appropriate width and slope that are accessible to and usable by individuals with disabilities (*37, 38*) must be provided in curbed areas that have sidewalks. For additional guidance, see the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*36*). Curb ramps are addressed in Section 4.17.3.

For further guidance on the accommodation of pedestrians, refer to the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*3*). Also, the FHWA publication *Informational Report on Lighting Designs for Midblock Crossings* (*18*) provides information on nighttime visibility concerns at nonintersection locations.

7.3.10 Provision for Utilities

The system of arterials in urban areas often serves as a utility corridor. Utilities should desirably be located underground or at the outer edge of the right-of-way. In addition, poles should be located as near the right-of-way lines as practical. Whenever practical, service access openings and covers should not be located in the traveled way but should preferably be placed outside the entire roadway. However, locations in the medians or parking lanes may be acceptable under special conditions. Utilities should seldom be added to an arterial by the open-cut method. Additional installations should be bored or jacked to avoid interference with normal traffic movements. The AASHTO *Guide for Accommodating Utilities within Highway Right-of-Way* (*4*) presents general principles for utility location and construction to minimize conflicts between the use of the right-of-way for vehicular movements and the secondary objective of providing space for locating utilities.

7.3.11 Intersection Design

The design and operation of intersections have a significant effect on the operational quality of an arterial. Intersection and stopping sight distance, pedestrian and bicycle movements, capacity, transit operations, grades, and provision for turning movements all affect intersection operation. Although encroachment of turning movements on adjacent lanes may be necessary in urban areas to avoid excessive corner radii (see the Section 9.6 discussion on effective turning radius design), the effects of such encroachments should be considered. Roundabouts are also becoming an increasingly popular intersection design alternative for many arterial intersections and should be considered in most design processes. It is recommended that each individual intersection be carefully evaluated in the early design phases. Chapter 9 discusses intersection development in detail.

7.3.12 Operational Control

The potential of traffic control measures to improve motor-vehicle capacity and level of service for all transportation modes should be exploited to the maximum degree on properly designed arterial streets while also considering context and the effects of these measures on other transportation modes. Improvements to the arterial system may help to relieve congestion on the local street system by diverting traffic to the more efficient and higher capacity arterial.

7.3.12.1 Traffic Control Devices

Where traffic signals are anticipated during the initial planning of an arterial street, intersection design should integrate the ultimate signal operation. The design should consider the reduction of signal phases by providing for concurrent opposing left-turn phases and by constructing left-turn lanes in a manner that will allow their free operation for all modes. Channelization, which provides for single or double left turns and free-flow right turns, often results in better signal control and may assist pedestrian crossings. However, multiple lane shifts to accommodate the installation of turn lanes should be designed in accordance with Sections 9.6.2 and 9.6.3.

Signal spacing to allow free-flow timing at a suitable operating speed in both directions of travel is highly desirable and may be achieved by controlling intersection locations during early land use development stages. If this cannot be achieved, suitable time-space diagrams based on traffic forecasts may be used to determine signal timing and spacing for major access points. Such efforts will allow optimum signal progression to provide maximum vehicle capacity and minimum vehicle delay time at speeds appropriate for the adjoining land uses. Driveway locations that unduly affect major through movements or interfere with the operation of an adjacent signalized intersection should be avoided. During the intersection design process, the physical location of signal supports can often be changed to reduce the potential for crashes involving vehicles that run off the road, to increase signal visibility, and to increase pedestrian accessibility.

The ultimate goal of any intersection design should be to serve the traffic demands of all users at a level and quality of service consistent with the overall arterial design and with as few crashes as practical. To accomplish this goal, all intersection elements, including traffic signals, should be integrated into all aspects of the design process. Traffic control devices such as signs, markings, signals, and islands are placed on or adjacent to an arterial to regulate, warn, or guide traffic. Each device is designed to fulfill a specific traffic control need. The need for traffic control devices should be determined by an engineering study conducted in conjunction with the geometric design of the street or highway. To provide uniform design and installation application of the various traffic control devices, refer to the current MUTCD (*12*).

Successful operation of an arterial in an urban area depends largely on proper pavement marking, especially on arterials having multiple lanes and particularly when special provision is made for left turns. Pavement marking materials that provide effective long-life markings should be used, even for areas where snow removal often obliterates ordinary markings in very short time periods. Overhead lane signing can be very helpful. Signs enable drivers to plan their maneuvers, and to change lanes where needed, well in advance of an intersection or decision point. Advance signs are especially helpful under adverse weather conditions, such as rain or snow. Adequate pedestrian crossing treatments and effective speed management enhance pedestrian movements on arterials in urban areas.

7.3.12.2 Provision and Management of Curb Parking

Curb parking reduces motor-vehicle capacity and creates friction with free flow of adjacent traffic, but in some contexts those effects are desirable to reduce through speeds and the provision of on-street parking provides an overall benefit. Replacing curb parking with through-travel lanes can increase the capacity of arterials with four- or six-lane curb-to-curb widths by 50 to 80 percent. On the other hand, in built-up areas, curb parking is often needed to sustain the viability of the community. Eliminating curb parking can reduce potential conflicts for pedestrians but can also reduce the livability of both commercial and residential districts.

Where parking provisions are included in the design, cross-sectional dimensions can be arranged such that the entire width can be used by moving traffic when parking is removed. At intersections, there should be a liberal distance from the corner of the intersection to the nearest parking stall. This distance should be at least 20 ft [6.0 m] from a crosswalk. This provides extra maneuvering space for turning traffic, reduces the conflict with through traffic, eliminates the need for parking vehicles to back across crosswalks, and increases sight distance. Where bulbouts, also called curb extensions, are utilized at intersections, the extended curb should extend down the roadway to the point where legal parking is resumed. This practice helps deter motorists from parking too close to the intersection.

While no other single operational control can have as dramatic an effect on traffic flow on arterial streets as the proper regulation of parking, indiscriminant parking bans can adversely affect the community through which the arterial street passes. Therefore, parking controls should be carefully considered, and where applied they should be vigorously enforced, particularly "No Parking" regulations in loading zones and at bus stops.

7.3.13 Speed Management in Design

In urban areas, the land use context and presence of nonmotorized users may suggest that an arterial be designed to effectively limit the resultant operating speeds on the facility to best balance the needs of all users. FHWA guidance states that "...in urban areas, the design of the street should generally be such that it limits the maximum speed at which drivers can operate comfortably, as needed to balance the needs of all users." In those situations, there are several choices in the selection of design elements and criteria for arterials in urban areas that can induce speed reductions and have other operational and crash reduction benefits for all road users. These include reduced lane widths, lane reductions, curb extensions, center islands or medians, on-street parking, and special intersection designs such as roundabouts. All of these speed management design techniques can be implemented on low-speed arterials and some may also be appropriate on high-speed roadways. For additional guidance on management of operating speed through the geometric design process, please refer to *Engineering Countermeasure for Reducing Speeds: A Desktop Reference of Potential Effectiveness* (*13*).

7.3.14 Directional Lane Usage

Typically, the conventional arterial street is a multilane two-way facility with an equal number of lanes for traffic in each direction of travel. Often, however, one-way operation is employed where conditions are suitable. Somewhat less frequently, reversible lane operation is used to improve operational efficiency. The conditions under which each form of operation is most suitable depend largely on traffic flow characteristics, street pattern, presence and activity of other modes, and geometric features of the particular street. Where a street system is undergoing expansion or improvement, the ultimate form of directional usage should be anticipated, and the design should be prepared accordingly. Once an arterial street is completed, conversion from one form of directional usage to another may involve considerable expense and disruption to traffic. For existing streets of conventional design, this conversion may be a practical alternative for increasing traffic capacity. For information concerning signing for directional lane usage, refer to the current MUTCD (*12*).

7.3.14.1 One-Way Operation

An arterial facility consisting of one or more pairs of one-way streets may be generally appropriate where the following conditions exist: (1) a single two-way street does not have adequate capacity and does not lend itself readily to improvement to accommodate anticipated traffic demand, particularly where left-turning movements at numerous intersections are difficult to handle; (2) there are two parallel arterial streets a block or two apart; (3) there are a sufficient number of cross streets and appropriate spacing to permit circulation of traffic; and (4) the amount of traffic recirculation caused by the one-way traffic pattern is not detrimental to the function of the land use context along or near the converted roadways.

One-way streets have the following advantages:

- Traffic capacity may be increased as a result of reduced midblock and intersection conflicts and more efficient operation of traffic control devices.
- Travel efficiency is increased as a result of reducing midblock conflicts and delays caused by slowing or stopped left-turning vehicles. The increase in the number of lanes in one direction permits ready passing of slow-moving vehicles. One-way operation permits good progressive timing of traffic signals.
- The number and severity of crashes is reduced by eliminating head-on crashes and reducing some types of intersection conflicts.
- Traffic capacity may be increased by providing an additional lane for through traffic. Although a two-way street with only one lane in each direction may not have sufficient width to accommodate two lanes in each direction, it may have sufficient width to accommodate three lanes in one direction when converted to one-way operation.
- The available street width is used fully through the elimination of need for a median.
- On-street parking that would otherwise have to be eliminated or curtailed may be retained.

Disadvantages to one-way operation are:

- Travel distances are increased because certain destinations can be reached only by around-the-block maneuvers. This disadvantage is more acute if the street grid is composed entirely of one-way streets.
- One-way streets may be confusing to drivers unfamiliar with the area.
- Emergency vehicles may be blocked by cars occupying all lanes at intersections while waiting for signals to change.
- Operating speeds may be higher than desired in comparison to similar two-way operations.

When considering a one-way street system, the operational disadvantages associated with one-way streets should also be considered. A one-way street system often forces drivers to take out-of-direction routes to their destinations, causing an increase in the volume of turning movements and the number of intersections a vehicle has to travel through. The direct result of this recirculation is an increase in vehicle miles traveled (VMT) and an increase in traffic volumes on a given segment or intersection within a one-way system. One-way systems generally yield 120- to 160-percent more turning movements when compared to a two-way system, and travel distances from a downtown entry point to destination is usually 20- to 50-percent greater in a one-way system when compared to a two-way system (*39*).

In summary, there are several possible advantages and disadvantages to one-way operation. The choice of one- or two-way operation depends largely on which type of operation can serve the traffic demands for all users most efficiently and with greatest benefit to the adjacent property and the context of the area. Both types of operation should be considered. In many cases, the proper choice is immediately obvious. In other instances, a thorough study involving all relevant considerations may be needed.

7.3.14.2 Reverse-Flow Operation

The familiar imbalance in directional distribution of traffic during peak hours on principal radial streets in large and medium-sized cities often results in congestion in the direction of heavier flow and excess capacity for opposing traffic. Capacity during peak hours can be increased by using more than half of the lanes for the peak direction of travel. In the application of reverse-flow operation, consideration should be given to the presence of other modes and the effect that such operation may have on those other modes.

Reverse-flow operation on undivided streets generally is justified where 65 percent or more of the traffic moves in one direction during peak periods, where the remaining lanes for the lighter flow are adequate for that traffic, where there is continuity in the route and width of street, where there is no median, where left turns and parking can be restricted, and where effects on all users are considered. Refer to the AASHTO *Guide for the Design of High Occupancy Vehicle Facilities* (*2*) for additional guidance concerning the appropriateness of reverse-flow operation.

The conventional undivided street need not be changed appreciably for conversion to reverse-flow operation, and the cost of additional control measures is not great. On a five-lane street, three lanes can be operated in the direction of heavier flow. On a six-lane street with directional distribution of approximately 65 to 35 percent, four lanes can be operated inbound and two lanes outbound during the morning peak. The assignment of the center lanes can be reversed during the evening peak so that two lanes are generated inbound and four lanes outbound. During off-peak periods, traffic is accommodated on three lanes in each direction or on two lanes in each direction with curb parking.

Streets with three or four lanes can also be operated with a reverse flow. However, with only one lane in the direction of lighter flow, a slow vehicle or one picking up or discharging a passenger will delay all traffic in that direction of travel, and a vehicle breakdown blocks traffic in that direction completely. Occasionally, circumstances may be such that such streets can be adapted to complete reverse flow (i.e., one-way inbound in the morning and one-way outbound in the evening). At other times the street may be operated as a two-way street, with or without parking.

Direct left turns by traffic in the off-peak direction on a two-way reversible street should be carefully controlled. Left turns from the predominant flow are subject to the same considerations and regulations as they are for conventional operation with two-way traffic. By contrast, on a one-way reversible street, left turns at all intersections can be readily made.

Reverse-flow operation needs special signing or additional control devices, or both. More policing and staffing are also needed to operate the control devices. Traffic cones or flexible tubes may be used to separate opposing traffic, and "No Left Turn" and "Keep Right" signs on pedestals or flexible posts are often used.

Assigning traffic to proper lanes can be accomplished by placing overhead signs indicating lane usage for specific times of day. These signs should be supplemented with traffic control signals located directly over each lane indicating when reversible lanes are open or closed to traffic in the specified direction. This is usually accomplished with a signal head displaying a red "X" for closed or a green directional down arrow for open. Refer to the MUTCD (*12*) for further guidance. This combination of signs and signals will decrease the undesirable potential for motorists to pull out for left turns into a lane that is signed for traffic in the opposite direction. It is better to place separate lane-use control signals at intervals over each lane. This method is particularly adaptable to long bridges and sections of streets without side connections.

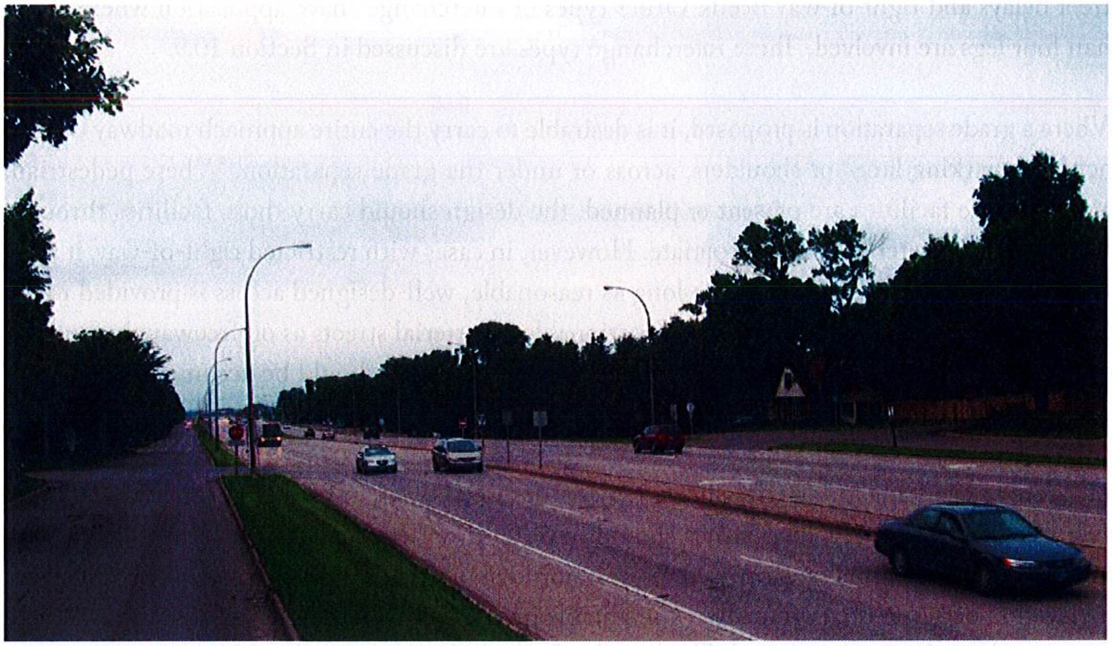
Further efficiency, as well as speed management, can be gained for the predominant direction of travel by progressive timing of signals. With an interconnected signal system, signals can be set for proper progression of the major movement in the peak periods. A third setting is used for the traffic flow during off-peak periods. In some cases, the signals for the center lane or lanes are set red in both directions during off-peak hours, thus converting the unused traveled way into a median area that separates traffic in opposite directions of travel and may, therefore, reduce crash frequency.

Reverse-flow operation on a divided facility is termed "contra-flow operation." While the principle of reverse-flow operation is applicable to divided arterials, the arrangement is more difficult than on an undivided roadway. The difficulty of handling cross and turning traffic, the potential confusion for pedestrians, and the potential for conflicts between opposing vehicles at high volumes may make other arrangements preferable to contra-flow operation. For example, the capacity of an undivided arterial with a reverse-flow lane allocation of three-two-three or three-three-three lanes (equivalent in peak-directional capacity to 10- or 12-lane conventional sections, respectively) may be comparable to the capacity of a six-lane freeway. For these widths, likely volumes would be 3,500 to 4,000 veh/h in one direction, or two-way ADT volumes of 50,000 to 60,000 vehicles per day, for which a freeway is warranted. Furthermore, traffic flows that are currently unbalanced may not remain unbalanced in future years. Reverse-flow operation for at-grade facilities is applicable chiefly as a means of increasing capacity on existing highways.

7.3.15 Frontage Roads and Outer Separations

Frontage roads are sometimes used on arterial streets to control access, to provide on-street parking, and to provide a more comfortable location for pedestrian and bicycle travel. Frontage roads are discussed in "Access Control through Geometric Design", Section 7.3.8.5. Other important functions of frontage roads are minimizing interference with operations on the through-traffic lanes while still providing access to abutting properties. For data on widths and other design features of frontage roads and outer separations, refer to Chapters 4, 9, and 10.

Figure 7-11 is an example of a two-way frontage road along a divided arterial with an appropriate distance from the edge of the arterial to the intersection of the cross street and frontage road in low-volume conditions. In moderate- to high-volume locations, moving the frontage road intersection away from the main intersection can provide additional space for vehicle storage between the intersections. Providing sufficient distance for turn-lane storage on the cross street is an important design feature in frontage road design.



Source: Minnesota DOT

Figure 7-11. Divided Arterial Street with Two-Way Frontage Road

7.3.16 Grade Separations and Interchanges

Grade separations and interchanges are addressed in Chapter 10 and many of the principles presented there are applicable to arterial streets. Although grade separations and interchanges are not often used on arterial streets due to high cost, limited right-of-way, increased operational challenges for pedestrians and bicyclists, and effects to frontage properties, they may be the only means available for providing sufficient capacity at some critical intersections.

In some cases, grade separations can be constructed within the existing right-of-way. Locations where grade separations could be considered on arterials in urban areas are:

- Very high-volume intersections between principal arterials
- High-volume intersections having more than four approach legs
- Arterial street intersections where all other principal intersections in the corridor are grade separated
- All railroad–highway grade crossings
- Sites where terrain conditions favor separation of grades

Normally, where a grade separation is provided on an arterial in an urban area, it is included as part of a diamond interchange. A single-point diamond interchange (SPDI) or diverging diamond interchange (DDI) can provide the benefits of a grade separation while reducing cross-street delays and right-of-way needs. Other types of interchanges have application where more than four legs are involved. These interchange types are discussed in Section 10.9.

Where a grade separation is proposed, it is desirable to carry the entire approach roadway width, including parking lanes or shoulders, across or under the grade separation. Where pedestrian and/or bicycle facilities are present or planned, the design should carry those facilities through and across the interchange as appropriate. However, in cases with restricted right-of-way, it may be appropriate to reduce the width so long as reasonable, well-designed access is provided to all user modes. Such a reduction is not as objectionable on arterial streets as on freeways because of lower speeds. The reduction in parking-lane or shoulder width should be accomplished with a taper. See Section 10.9.6 for a discussion about taper design elements.

Interchange elements for arterial streets may be designed with lower dimensional values than with freeways. Desirably, loop ramps should have radii no less than 100 ft [30 m]. Diamond ramps may have lengths as short as the minimum distance needed to overcome the difference in elevation between the two roadways at suitable gradients and to accommodate traffic storage queue needs at the ramp terminal. The length of speed-change lanes should be consistent with design speed. Chapter 10 provides criteria for design of interchanges and grade separations.

7.3.17 Erosion Control

When an arterial in an urban area is designed with an open-ditch cross section, rural erosion control measures should be applied and water quality effects should be considered. Curbed cross sections usually need more intensive treatment to prevent damage to adjacent property and siltation in sewers and drainage systems. Seeding, mulching, and sodding are usually employed to protect disturbed areas from erosion. Landscaping features, such as groundcover plantings, bushes, and trees also control erosion, enhance beauty, and provide a visual buffer for adjacent properties.

7.3.18 Lighting

Adequate lighting can be very important to provide visibility for all users and reduce crash frequencies on an arterial in an urban area at night and can also aid older drivers. The higher volumes and speeds that are typically found on arterials make it especially challenging for the driver to make correct decisions with adequate time to execute the proper maneuvers without creating undue conflict in the traveled way. Pedestrian and bicycle movements, along with transit access, are also made more challenging by higher volumes and speeds on arterial roadways. Where lighting is adequate, sudden braking and swerving are minimized and visibility of nonmotorized users is improved. The visibility of signing and pavement marking also helps to smooth traffic flow. A well-designed, adequate lighting system is more important to optimum operation for all users of an arterial in an urban area than for any other type of city street. The lighting should be continuous and of an energy-saving type. Lighting in an urban area is often a matter of civic pride and is a deterrent to crime. In the event that it is impractical to provide continuous lighting, consideration should be given to providing intermittent lighting at such locations as intersections, areas of high pedestrian and/or bicycle activity, and ramp termini. Additional or special lighting may also be needed on roadway borders to illuminate separated sidewalks and bicycle or multi-use paths. The AASHTO *Roadway Lighting Design Guide* (*5*), FHWA *Lighting Handbook* (*22*) and ANSI/IESNA RP-8 *American National Standard Practice for Roadway Lighting* (*30*) are recommended as sources of lighting information.

7.3.19 Public Transit Facilities

Wherever there is a demand for arterials to serve passenger car traffic, there is likewise a potential demand for public transportation. With increasing use of fixed-rail transit vehicles in surface streets and the increased use of free-wheeled buses, public transit is often an important consideration in design of arterials in urban areas. Other high-volume passenger vehicles such as the minibus, taxicab, and limousine may merit serious consideration in the overall planning of a high-volume arterial. The transit vehicle is more efficient than the private automobile with respect to street space occupied per passenger carried. With proper recognition of transit needs and provisions for them in the design and operation of arterials, buses can become even more compatible with arterial traffic in the future. The detailed discussion of bus facilities presented below is not intended to limit consideration of other types of mass transit facilities. The more sophisticated public transit modes such as streetcars, trolleys, and fixed rail, present unique and varied challenges that are often difficult to integrate with roadway design. To address this need, AASHTO has developed the *Guide for Geometric Design of Transit Facilities on Highways and Streets* (*8*). The discussion below concentrates on the transit arrangements that most often affect arterial roadway design, namely bus transit.

The vehicle-carrying capacity of through-traffic lanes is typically decreased when a transit vehicle and other traffic use the same lanes. A bus stopping for passenger loading, for example, not only blocks traffic in that particular lane but affects traffic operations in other lanes. It is desirable that such interferences be minimized through careful facility planning, design, and traffic control measures.

The needs of public transit should be considered in the development of an urban highway improvement program. The routings of transit vehicles (including turns and transfer points) and the volumes of buses (i.e., average or minimum headways) and passenger loading/unloading should be considered in highway design. Design and operational features of the highway that are affected by these considerations include: (1) locations of bus stops (spacing and location with respect to intersections and pedestrian crosswalks), (2) design of bus stops and turnouts, (3) reservation of bus lanes, and (4) special traffic control measures. Because some of the design and control measures that are beneficial to bus operation have an adverse effect on other traffic, and vice versa, a compromise that is most favorable to all users is appropriate.

7.3.19.1 Location of Bus Stops

The demand for bus service is largely a function of land-use patterns. The general location of bus stops is largely dictated by patronage and by the locations of intersection bus routes and transfer points. Bus stops should be located primarily for the convenience of patrons. The geometric design process should provide for appropriate pedestrian and bicycle access to those stop locations.

The specific location of a bus stop within the general area where a bus stop is needed is influenced not only by convenience to patrons but also by the design and operational characteristics of the highway. Except where cross streets are widely spaced, bus stops are usually located in the immediate vicinity of intersections. This facilitates crossing of streets by patrons without the need for midblock crosswalks. Midblock locations for bus stops may be appropriate where blocks are exceptionally long, or where bus patrons are concentrated at places of employment or residences that are well removed from intersections. Midblock bus stops will generally need provision for midblock pedestrian crossings.

Bus stops at intersections may be located on the near (approach) or far (departure) side of the intersection. Although there are advantages and disadvantages to both near- and far-side locations, in most cases far-side locations are preferred. However, the specific location for each bus stop should be examined separately to determine the most suitable arrangement. Factors for consideration include service to bus patrons, efficiency of transit operations, and efficiency of traffic operations. Far-side bus stops are advantageous at intersections where (1) other buses may turn left or right from the arterial; (2) turning movements from the arterial by other vehicle types, particularly right turns, are heavy; and (3) approach volumes are heavy, creating a large demand for vehicle storage on the near-side approach. Far-side bus stops have also proven to be effective in reducing collisions involving pedestrians. Sight distance conditions generally favor far-side bus stops, especially at unsignalized intersections; a driver approaching a cross street on the through lanes of an arterial can better see any vehicles approaching from the right if no bus is present. At near-side bus stops, the view of through drivers to their right may be blocked by a stopped bus. If the intersection is signalized, the bus may block the view of one of the signal heads.

Another disadvantage of near-side bus stops is the difficulty encountered by other vehicles in making turns while a bus is loading. Drivers frequently proceed around the bus to turn right, which interferes first with other traffic on the arterial and then with the bus as it leaves the stop. This disadvantage is eliminated if the cross street is one way from right to left. Thus, where the street pattern consists of a one-way grid, there is some advantage in having stops at alternate cross streets in advance of the streets crossing from right to left.

Where buses turn left at an intersection, the bus stop in advance of the intersection should be located at least one block before the turn, and the next bus stop should be located on the intersecting street after the turn is completed. Even with this arrangement, the bus will need to cross all traffic lanes in the direction of travel to reach the left lane for the turn.

On highly developed arterials with ample rights-of-way, bus turnouts, and speed-change lanes, there is a definite traffic advantage to the far-side bus stop. Such stops can be combined with speed-change lanes for turning vehicles entering the arterial. Where the stop is located on the near side of an intersection, vehicles turning right from the through lanes of the arterial cannot use the deceleration lane when it is occupied by a transit vehicle and instead may maneuver around it on the through lanes. Where the bus stop is located on the far side of the intersection, traffic turning right from the arterial does so freely.

On an arterial with frontage roads, buses may leave and return to the arterial by special openings in the outer separation in advance of and beyond the intersection. This arrangement has the advantage that buses stop in a position that is well removed from the through lanes. Right-turning traffic to and from the arterial street may also use these special openings, thereby reducing conflicts at the intersection proper. In an alternate arrangement, no slot in advance of the intersection is provided, and buses can cross to the frontage road at the intersection proper. Both slots may be eliminated where the frontage road is continuous between successive cross streets because buses can leave the through lanes at one intersection and use the frontage road to reenter the arterial at the next intersecting street. This type of operation is fitting where bus stops are widely spaced.

Midblock bus stops, like far-side stops, have an advantage over near-side stops in that the full roadway width on the intersection approach is made available for vehicle storage and turning maneuvers to maintain capacity as high as practical. However, midblock bus stops are not generally suitable for streets where parking is permitted, as is the case on some arterials during off-peak hours. Usually, a crosswalk is needed at midblock bus stops to provide access to the stops from either side of the arterial and to serve as an intermediate crosswalk for other pedestrian traffic. Where the pedestrian crossing demand and traffic volumes are high, signal control may be needed to create crossing opportunities for pedestrians. Midblock signals violate driver expectations and should generally be used only where pedestrian crossing demand indicates a clear need. At a major transit stop with heavy pedestrian movements, a pedestrian grade separation may be warranted.

Additional information concerning the location and design of bus stops is presented in TCRP Report 19, *Guidelines for the Location and Design of Bus Stops* (*33*) and the AASHTO *Guide for Geometric Design of Transit Facilities on Highways and Streets* (*8*).

7.3.19.2 Bus Turnouts

The interference between buses and other traffic can be considerably reduced by providing stops clear of the lanes for through traffic. However, since bus operators may not use the turnout if they have difficulty maneuvering back into traffic, the bus turnout should be designed so that a bus can enter and leave easily. The preceding discussion illustrates methods for reducing interference between buses and through traffic on higher-speed arterials. For geometric details, see Section 4.19 on "Bus Turnouts" and the AASHTO *Guide for Geometric Design of Transit Facilities on Highways and Streets* (*8*). It is somewhat rare that sufficient right-of-way is available on lower-speed arterial streets to permit turnouts in the border area, but for streets with on-street parking, judicious use of parking restrictions can provide the same benefits.

7.3.19.3 Reserved Bus Lanes

Some improvement in transit service can be realized by excluding other traffic from selected lanes of arterial streets, particularly curb lanes in the urban core context. The success of this regulatory measure is rather limited in most instances, however, because vehicles making right turns occupy this same lane, it is not practical to exclude them, for distances up to a block or two in advance of the turn. Vehicles preparing to turn right cannot be distinguished from through traffic, so compliance with the exclusive bus lane regulation is largely on a voluntary basis. Nevertheless, there are certain combinations of conditions under which at least a modest improvement in transit service can be achieved. These conditions are not always apparent or definable, and the only way to determine conclusively that there will be overall benefit is to test the regulation in practice at locations where a preliminary investigation indicates likelihood of success. Figure 7-12 shows a typical reserved bus lane for peak-hour use.

Figure 7-12. Reserved Bus Lane Source: MRIGlobal

7.3.19.4 Traffic Control Measures

Traffic control devices on arterial streets are usually installed with the intent of favoring automobile traffic, with only secondary consideration to transit vehicles. For express-bus or bus rapid transit operation, the control measures that are most favorable for one mode will generally be equally well-suited for the other. However, where local service is provided by buses with frequent stops to pick up and discharge passengers, a signal system that provides for good progressive movement of privately operated vehicles may actually result in reverse progression for buses. The resulting slow travel speed for buses tends to discourage patronage, further increasing the already heavy volume of automobile traffic.

Traffic control systems have been developed that are more favorable for bus service without serious adverse effects on other traffic. This approach holds some promise of improving average travel speeds for buses and making public transit more attractive. One method of prioritizing bus movements without reducing travel speeds for passenger cars is by extending the green time for an approaching bus so the bus can clear the intersection and then load and unload on the far side while the light is red. Other techniques involve providing an exclusive advanced green signal for transit vehicles so they may proceed through an intersection before regular traffic is released. Development of a suitable signal system operation involves careful investigation by properly skilled personnel and should be a part of an arterial improvement program that involves the joint efforts of traffic specialists, the transit industry, and the design team.

Although the major emphasis in the application of traffic control measures is in minimizing delay, the control measures can facilitate bus operation in other respects, particularly where buses turn from the arterial onto a cross street.

Buses making right turns may create a problem where the cross street is narrow and the adjoining property is developed so intensively that it is not practical to provide a sufficiently long curb return radius. Buses turning right from the curb lane may encroach beyond the centerline of the cross street. At signalized intersections, the space beyond the centerline is normally occupied by vehicles stopped for the red signal. Under such conditions, the stop line on the cross street should be set back to provide sufficient space for turning maneuvers by buses. If needed, an auxiliary signal head could be placed at the relocated stop line to obtain compliance.

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Chapter 2 – Appendix 4

Basic Considerations or Starting Points in Design for Local Government Structures

(September 2019)

<u>Basic considerations or starting points in design for local government structures</u>
September 2019

Prestressed Girder Bridges (PGB)

- Most local government bridges usually have a 28'-30' roadway width requiring a minimum number of girders. (SDDOT requires at minimum a 4-girder system – good design starting point for the roadway widths noted). Girder depth and configuration should be optimized based on roadway width and span length requirements.
- Decks can usually be designed efficiently with an 8.25" deck thickness. Following AASHTO Code requirements one can optimize the deck thickness to minimize concrete quantities.
- Bents – Optimize the number of columns in bents. Generally speaking, 2 column bents will be adequate for the roadway widths used on LGA structures, however skewed structures or long span structures may require more columns.
- Integral abutments – Use a 3' pile cap with a minimum grout pad height above the construction joint of 1.5". Grout pads in excess of 6" in height should be reinforced, however, given the relatively narrow bridges used by LGA's, this should rarely be encountered. In most cases if following the above guidelines, the integral abutment assumptions should be valid and frame action calculations will not be required.
- Due to constructability issues with 27 M prestressed girders, SDDOT does not recommend their use.
- Wing configurations shall be proportioned to minimize wing length as well as keep the embankment spill cone from contacting the superstructure. Use a 2:1 inslope, or that required to meet clear zone/guard rail requirements, immediately adjacent to the abutment wing and carry the berm slope to the top of the wing when calculating wing length. Check to ensure that the spill cone does not interfere with superstructure elements. SDDOT would like to see layout calculations for abutments wings.

Cast-in-place and Precast Reinforced Concrete Box Culverts

- For boxes 8' in height or less, use 7" walls and increase the wall thickness by ½" per 1' of height after 8' unless by structural analysis thicker walls are needed. The guidance noted is for constructability and has proven to be adequate over many years of box culvert construction.
- When box cell spans exceed 12' consider adding another cell. The slab thicknesses on box culvert spans exceeding 12' start to get thick and may be less economical. A cost estimate should be prepared to determine the most economical box configuration.
- Maximum skew allowed for precast box culverts is 10 degrees. Skews beyond this limit require additional barrel length resulting in increased costs and standard precast inlet and outlet wing configurations result in slopes adjacent to the wings steeper than 2:1.
- On shallow fills (approximately 3' or less on asphalt or gravel roadways) where clear zone controls the length, the parapets on box culverts shall be placed outside the clear zone. Where fill controls the length of a box culvert, calculate the length of the barrel required assuming a 6"-9" dirt pile-up at the back of the parapet. For shallow fills of 3' or less where the clear zone controls length, the box will be designed for 0 to 5' fill. Boxes with deeper fills or under paved roadways, where there is little to no chance that maintenance forces will remove material from over the box, may be designed for minimum fills of 1' or 2', as appropriate.
- The bottom slab thickness shown on the plans should be the same as the top slab unless structural analysis deems it necessary to be thicker. The structural model used for analysis should use a bottom slab thickness 1" less than detailed on the plans. The intent is that 1" shall be added to design bottom slab thickness to allow for irregularities in the ground surface; however, it is not intended that this extra inch be included in the structural model. For example, a box culvert structure may be modeled with a 12" top slab and an 11" bottom slab. If

this is structurally acceptable, the top and bottom slabs would be detailed as 12", effectively adding 1" to the bottom slab to allow for any irregularities in the ground surface. Maximum rebar size shall be a No. 8.

- Riprap adjacent to inlet and outlet box culvert wings on the inslope is used when H&H analysis indicates a need or when there is evidence of significant erosion at either routine NBI inspections or at the TS&L inspection.
- All cast-in-place box culverts shall have reinforced concrete aprons (inlet and outlet) with #4 rebar at 12" spacing.
- The SDDOT requests that wingwall layout calculations be submitted along with the structure design calculations. Wingwall lengths can vary depending on skew, fill height, clear zone, etc.

Double Tee Bridges/Bulb Tee Bridges

- Berm type abutment designs are preferred.
- Pay attention to camber when using these beams in straight or sag vertical curves. Rideability has been an issue in the past.
- Single span structures tend to work the best.
- Maximum 65' spans for Dbl T's and 90' spans for Bulb T's – Check with suppliers for maximum length design and availability

Continuous Concrete Bridges (CCB's)

- Structures skewed from 20-30 degrees shall be designed for the span length perpendicular to the supports. Need to note how reinforcing is placed and the design considerations that steel configuration requires. Also note that edge beams must be designed for span lengths parallel to centerline and railing loads are distributed into the deck perpendicular to centerline roadway.
- Span ratio shall be kept at 1.25:1 to optimize slab thickness for bending moment and keep dead load moment induced into intermediate supports to a minimum.
- Wing configurations shall be proportioned to minimize wing length as well as keep the embankment spill cone from contacting the superstructure. Use a 2:1 inslope, or that required to meet clear zone/guard rail requirements, immediately adjacent to the abutment wing and carry the berm slope to the top of the wing when calculating wing length. Check to ensure that the spill cone does not interfere with superstructure elements. SDDOT would like to see layout calculations for abutments wings.

Precast Concrete Arch structures

- Due to scour concerns, precast concrete arch structures are generally only allowed when bedrock is near the surface If used, place on cast-in-place foundations founded on bedrock at or near (within a foot or two) the flowline of the stream.

Rigid frame structures

- The superstructure/abutment/abutment wing configuration must be correctly modeled and designed to accommodate structure response to force effects.

Chapter 2 – Appendix 5

AASHTO “*Guidelines for Geometric Design of Low-Volume Roads*” (Second Edition, 2019)

SECOND EDITION

Guidelines for Geometric Design of Low-Volume Roads

2019

American Association of State Highway and Transportation Officials

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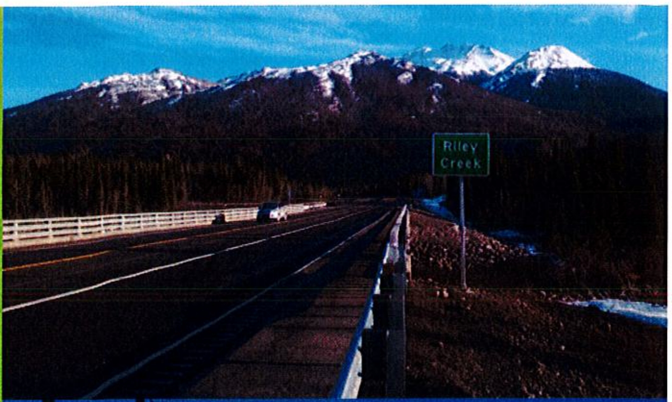



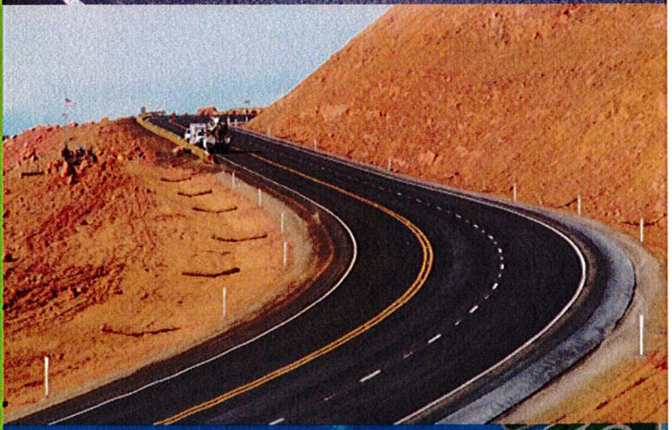
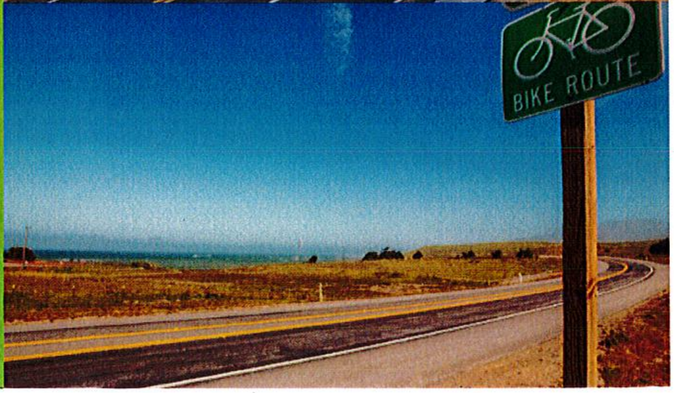

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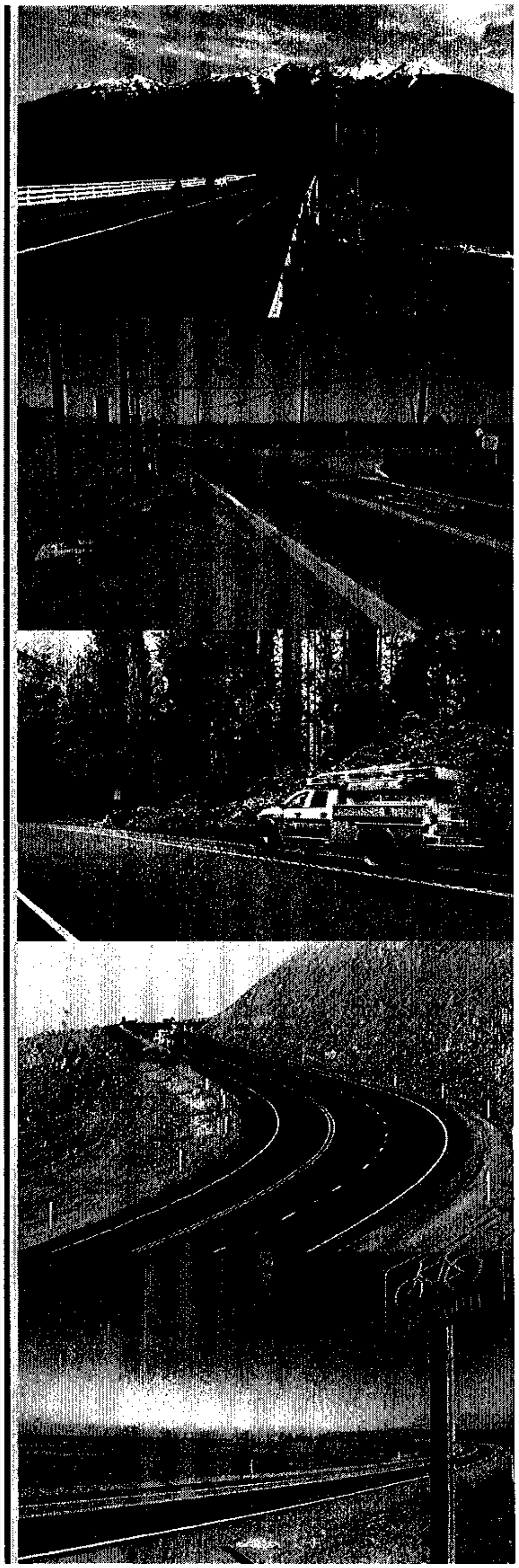

American Association of State Highway and Transportation Officials
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FOREWORD

As road designers, engineers strive to provide for the needs of highway users while maintaining the integrity of the environment. Unique combinations of design requirements that are often conflicting result in unique solutions to design problems. The geometric design of low-volume roads presents a unique challenge because the low traffic volumes and reduced frequency of crashes make designs normally applied on higher volume roads less cost effective. The guidance supplied by this document, *Geometric Design Guidelines for Low-Volume Roads*, addresses the unique needs of such roads and the geometric designs appropriate to meet those needs. These guidelines may be used in lieu of the guidance in *A Policy on Geometric Design of Highways and Streets*, commonly known as the Green Book (**5**).

The first edition of these guidelines, published by AASHTO in 2001, which addressed roads with design volumes of 400 vehicles per day or less, was the result of a research and development process initiated by AASHTO in 1996. These guidelines were initially developed through two projects of the National Cooperative Highway Research Program (NCHRP), which is jointly sponsored by AASHTO and the Federal Highway Administration. After completion of the NCHRP research, these guidelines went through the normal AASHTO review process. During the development process of the first edition, representatives of other interested organizations such as the National Association of County Engineers, the American Society of Civil Engineers, the U.S. Forest Service, the American Public Works Association, and the National League of Cities participated in the review of the guidelines.

This second edition is the result of a new research and development process initiated by AASHTO in 2013 through the NCHRP program. After completion of the NCHRP research, these guidelines went through the normal AASHTO review and balloting process. AASHTO appreciates the advice of the U.S. Forest Service in the development of this edition of the guidelines. The scope of these new guidelines includes local and minor collector roads with traffic volumes of 2,000 vehicles per day or less.

Design values are presented in this document in both U.S. customary and metric units and were developed independently within each system. The relationship between the U.S. customary and metric values is neither an exact (soft) conversion nor a completely rationalized (hard) conversion. The U.S. Customary values are those that would have been used had the policy been presented exclusively in U.S. Customary units; the metric values are those that would have been used if the policy had been presented exclusively in metric units. Therefore, the user is advised to work entirely in one system and not attempt to convert directly between the two.

The fact that new design values are presented herein does not imply that existing streets and highways are unsafe, nor does it mandate the initiation of improvement projects. *A Policy on Geometric Design of Highways and Streets* (**5**) states that specific site investigations and crash history analysis often indicate that the existing design features are performing in a satisfactory manner. The cost of full reconstruction for these facilities, particularly where major realignment is not needed, will often not be justified. This is especially true for low-volume roads, which experience substantially fewer crashes than roads with design volumes greater than 2,000 vehicles per day. These guidelines recommend an approach to geometric design for low-volume roads, including both new construction and projects on existing roads, that is based on research concerning the safety cost-effectiveness of geometric elements. For projects on existing roads, reviews of site-specific conditions also are a key element of the design guidance presented here.

These guidelines address issues for which appropriate geometric design guidance for low-volume roads differs from the policies normally applied to higher volume roads. For any geometric design issues not addressed by these guidelines, design professionals should consult *A Policy on Geometric Design of Highways and Streets* (**5**).

The intent of these guidelines is to assist the designer by referencing a recommended range of values for critical dimensions. It is not intended to be a detailed design manual that could supersede the need for the application of sound principles by the knowledgeable design professional. Flexibility in application of these guidelines is encouraged so that independent designs tailored to particular situations can be developed.

Roads, vehicles, drivers, and nonmotorized users (pedestrians and bicyclists) are all integral parts of transportation safety and efficiency. While this document primarily addresses geometric design of roads, a properly equipped and maintained vehicle and reasonable and prudent performance by the user are also needed for safe and efficient operation of the transportation facility.

PREFACE

This Guideline was developed as part of the continuing work of the AASHTO Council on Highways and Streets. The Council, then titled the Committee on Planning and Design Policies, was established in 1937 to formulate and recommend highway engineering policies. This Council has developed *A Policy on Geometric Design of Rural Highways*, 1954 and 1965 editions; *A Policy on Arterial Highways in Urban Areas*, 1957; *A Policy on Design of Urban Highways and Arterial Streets*, 1973; *Geometric Design Standards for Highways Other Than Freeways*, 1969; *A Policy on Geometric Design of Highways and Streets*, 1984, 1990, 1994, 2001, 2004, and 2011; *A Policy on Design Standards—Interstate System*, 1956, 1967, 1988, 1991, 2005, and 2016; and a number of other AASHTO policy and guide"publications.

An AASHTO publication is typically developed through the following steps: (1) The Council selects subjects and broad outlines of material to be covered. (2) The appropriate subcommittee and its task forces, in this case the Committee on Design and the Technical Committee on Geometric Design, assemble and analyze relevant data and prepare a tentative draft. Working meetings are held and revised drafts are prepared, as necessary, and reviewed by the Committee until agreement is reached. Standards and policies must be adopted by a two-thirds vote by the Member Departments before publication. During the developmental process, comments are sought and considered from all the states, the Federal Highway Administration, and representatives of the American Public Works Association, the National Association of County Engineers, the National League of Cities, and other interested parties.

This Guideline was first published by AASHTO in 2001 for application to very low-volume local roads and some collector roads with design average daily traffic volumes of 400 vehicles per day or less. This second edition has been updated for application to all low-volume local and minor collector roads with design average daily traffic volumes of 2,000 vehicles per day or less.

1 Introduction

1.1 INTRODUCTION

This document presents geometric design guidelines for low-volume roads. The purpose of the guidelines is to help highway designers in selecting appropriate geometric designs for roads with low traffic volumes. The design guidelines presented here may be used on low-volume local and minor collector roads in lieu of the applicable policies presented in the AASHTO publication, *A Policy on Geometric Design of Highways and Streets* (**5**), commonly known as the Green Book.

This chapter defines low-volume roads, describes the scope of the design guidelines, explains the relationship of the guidelines to other AASHTO policies, and presents the organization of the remainder of this document.

1.2 DEFINITION OF LOW-VOLUME ROADS

The guidelines presented in this document are applicable to low-volume roads. Low-volume roads are defined as follows:

A *low-volume road* is a road that is functionally classified as a local or minor collector road and has a design average daily traffic volume of 2,000 vehicles per day or less.

The preceding statement indicates that the functional classification of a road is a key element of the definition of a low-volume road in these guidelines. A *local road* is a road whose primary function is to provide access to residences, farms, businesses, or other abutting property, rather than to serve through traffic. Although some through traffic may occasionally use a local road, through traffic service is not its primary purpose. The term local road is used here to refer to the functional classification of the road and is not intended to imply that the road is necessarily under the jurisdiction of a local or municipal unit of government. *Rural minor collector roads* generally serve travel of intracounty rather than statewide importance and constitute those routes on which predominant travel distances are shorter than major collector and arterial routes. *Urban minor collector streets* provide both land access service within residential neighborhoods and commercial and industrial areas and connections to streets of higher functional classification. Administrative arrangements for operation of the highway system vary widely and, in different parts of the United States, roads that are functionally classified as local and minor collector roads may be under Federal, state, or local control.

More than 80 percent of the roads in the United States have traffic volumes of 2,000 vehicles per day or less. The low-volume local and minor collector roads, defined above, to which the guidelines presented in this document are applicable, should include most of this extensive road mileage. Little of this road mileage would be classified as arterials. In some states, portions of the state-numbered route system meet the definition of low-volume minor collector roads and can be addressed with these guidelines.

In addition to serving motor vehicle traffic, low-volume roads also serve varying numbers of pedestrians and bicyclists. The needs of pedestrians and bicyclists should be considered in the design of each project. Some low-volume roads may serve pedestrians and bicyclists only rarely, but other low-volume roads, particularly in urban areas, serve pedestrians or bicyclists, or both, in sufficient numbers that specific pedestrian or bicycle facilities, or both, are needed.

1.3 SCOPE OF GUIDELINES

The guidelines presented in this document are intended for application in the design of low-volume roads, as defined above, including application in new construction of low-volume roads and in the improvement of existing low-volume roads. The scope of the guidelines includes roads in both rural and urban areas.

These design guidelines enable designers for projects on low-volume roads to apply design criteria that are less restrictive than those generally used on *higher volume roads*. Where the term *higher volume roads* is used in this document, this refers to roads with design volumes of more than 2,000 vehicles per day, which are outside the scope of these guidelines. The risk assessment on which the guidelines are based shows that these less restrictive design criteria can be applied on low-volume roads without substantial effects on crash frequency and severity. The guidelines discourage widening of lanes and shoulders, changes in horizontal and vertical alignment, and roadside improvements except in situations where such improvements are likely to provide substantial reductions in crash frequency or severity. Thus, projects designed in accordance with these guidelines are less likely to negatively impact the environment, roadway and roadside aesthetics, existing development, historic and archeological sites, and endangered species. In reviewing the geometric design for sections of existing roadway, designers should strive for consistency of design between that particular section and its adjoining roadway sections. The potential effects of future development that may affect the traffic volume, vehicle mix, and presence of pedestrians or bicyclists on the roadway should also be considered.

These design guidelines are intended to encourage rational safety management practices on low-volume roads. Expenditures for highway improvements are discouraged at sites where such improvements are likely to have little effect on crash frequency or severity, but are strongly encouraged at sites where crash patterns exist that can be corrected by a roadway or roadside improvement. Designers are provided substantial flexibility to retain the existing roadway and roadside design, where that existing design is performing well, but are also provided flexibility to recommend improved designs, even designs that exceed the guidelines presented here, where needed to correct documented crash patterns or meet other agency goals.

The scope of these guidelines includes geometric design for new construction and for improvement of existing roads. Geometric design criteria for new construction apply to construction of a new road where none existed before. Projects on existing roads may involve reconstruction, resurfacing, rehabilitation, restoration, and other types of improvements.

These guidelines are limited in scope to geometric design issues and do not address the application of traffic control devices on low-volume roads. For traffic control device guidance, see the *Manual on Uniform Traffic Control Devices* (**8**).

1.4 RELATIONSHIP TO OTHER AASHTO POLICIES

The design guidelines presented in this document may be applied to low-volume roads in lieu of the applicable policies of the AASHTO Green Book (**5**) and the AASHTO *Roadside Design Guide* (**3**). For projects on local roads and streets, these design guidelines may be applied in place of Chapter 5 (Local Roads and Streets) of the AASHTO Green Book (**5**) to local roads that serve design volumes of 2,000 vehicles per day or less. For projects on minor collector roads and streets that serve design volumes of 2,000 vehicles per day or less, these design guidelines may be applied in place of the applicable policies in Chapter 6 (Collector Roads and Streets) of the AASHTO Green Book (**5**). The design guidelines presented here address design issues for which an explicit risk assessment has been performed. For design issues that are not addressed in these guidelines, the designer should consult the applicable sections of the AASHTO Green Book (**5**) and the AASHTO *Roadside Design Guide* (**3**). Design of facilities for nonmotorized users is addressed in the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (**1**) and the AASHTO *Guide for the Development of Bicycle Facilities* (**4**).

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2 Framework for Design Guidelines

2.1 INTRODUCTION

This chapter presents a framework for the design guidelines for low-volume roads. The elements of this framework are area type, roadway functional class and subclass, design speed or operating speed, and design volume. The chapter identifies how these elements of the framework are used in identifying the appropriate design guidelines for a specific design application.

2.2 AREA TYPE

The design guidelines are applicable to both rural and urban areas. The operating characteristics, constraints, and configurations of low-volume roads in rural and urban areas differ substantially and, therefore, in many cases, the design guidelines for rural and urban roads also differ. Thus, before applying the design guidelines, the designer should determine the area type in which the site of interest is located.

Low-volume roads in rural areas are more likely than urban roads to operate at high speeds and have a cross section with open drainage (shoulders and ditches, rather than curb and gutter). Rural roads tend to have fewer right-of-way constraints, less pedestrian activity, and a broader range of uses than urban roads. Low-volume rural roads include roads in undeveloped or agricultural areas as well as roads in rural towns.

By contrast, urban and suburban roads, even those with low traffic volumes, are generally more constrained than rural roads in terms of speeds and right-of-way. The guidelines for urban roads presented in this document apply to both urban and suburban conditions.

2.3 FUNCTIONAL CLASSIFICATION

The concept of functional classification is fundamental to the criteria used in the geometric design of highways and streets. The functional classification of a roadway identifies the relative importance of the mobility and access functions for that roadway. Roadways in the highest functional class are freeways. Freeways are intended primarily to serve through traffic traveling relatively long distances and provide no access to adjacent land except by way of interchanges spaced at appropriate intervals. Arterials and major collectors provide progressively less emphasis on mobility for through traffic and more emphasis on access to adjacent land. Local roads are intended to provide access to residences, businesses, farms, or other abutting property and are not intended to serve through traffic, although a limited amount of through traffic may use some local roads.

The design guidelines presented in this document are applicable to local and minor collector roads with design volumes of 2,000 vehicles per day or less. Some highway agencies distinguish between major and minor collectors only in rural areas, although Federal guidelines now permit this distinction in urban areas as well. If an agency does not formally use the minor collector classification in urban areas, designers should exercise judgment in applying the guidelines to low-volume roads that are classified as collectors and are closer in character to local roads than to arterials.

For purposes of the design guidelines, low-volume roads are further subdivided into six functional subclasses for rural facilities and three functional subclasses for urban facilities as follows:

Rural Roads:

- rural major access roads,
- rural minor access roads,
- rural industrial or commercial access roads,
- rural agricultural access roads,
- rural recreational and scenic roads, and
- rural resource recovery roads.

Urban Roads:

- urban major access streets,
- urban residential streets, and
- urban industrial or commercial access streets.

Each of these functional subclasses is defined below.

Figures 2-1 and 2-2 present photographs of typical rural and urban low-volume roads.

Figure 2-1. Typical Rural Low-Volume Roads (Source: CH2M Hill/Jacobs Engineering)

Figure 2-2. Typical Urban Low-Volume Streets (Source: MRIGlobal)

2.3.1 Rural Major Access Roads

Rural major access roads serve a dual function of providing access to abutting properties as well as providing through or connecting service between other local roads or higher type facilities. In rural areas, major access roads may have significant local continuity and may operate at relatively high speeds. Because of the possibility of through traffic, there may be a meaningful segment of traffic that includes unfamiliar drivers. Major access roads classified as local roads may thus, in some respects, function like collector or even minor arterial roads. Major access roads are usually paved, but may be unpaved in some rural areas. Minor collector roads should be treated as major access roads for purposes of these guidelines. Traffic on rural major access roads is largely composed of passenger vehicles or other smaller vehicle types. However, such roads need to be accessible to school buses, fire trucks, other emergency vehicles, and maintenance vehicles such as snow plows and garbage trucks. Access roads serving commercial or industrial land uses are described below in a separate functional subclass.

2.3.2 Rural Minor Access Roads

Rural minor access roads serve almost exclusively to provide access to adjacent property. Many of these roads are cul de sacs or loop roads with no through continuity. The length of minor access roads is typically short. Because their sole function is to provide access, such roads are used predominantly by familiar drivers.

Minor access roads generally serve residential or other non-commercial land uses. Speeds are generally low for the local environment, given the purpose of the road and short trip lengths. As noted above, many minor access roads end in cul de sacs or dead ends, thus limiting the opportunity for high travel speeds. Minor access roads are frequently narrow, and in some rural areas may function as one lane roads. Minor access roads can be either paved or unpaved. Traffic is largely composed of passenger vehicles or other smaller vehicle types. However, such roads need to be accessible to school buses, fire trucks, other emergency vehicles, and maintenance vehicles such as snow plows and garbage trucks. Access roads serving commercial or industrial land uses are described below in a separate functional subclass.

2.3.3 Rural Industrial or Commercial Access Roads

Industrial or commercial access roads serve developments that may generate a significant proportion of truck or other heavy vehicle traffic. The primary or sole function of such roads is generally to provide access from a factory or another commercial land use to the local or regional highway network. Typical industrial or commercial access roads are very short and, in many cases, they do not serve any through traffic. Industrial or commercial access roads may be either paved or unpaved. Such industrial or commercial access roads are addressed in their own functional subclass, separately from minor access roads, which they otherwise resemble, because consideration of trucks and other heavy vehicles is important in their design.

2.3.4 Rural Agricultural Access Roads

Certain roads in rural areas serve primarily to provide access to fields and farming operations. Vehicle types that use such roads include combines, tractors, trucks that haul agricultural products, and other large and slow-moving vehicles with unique operating characteristics. The driving population generally consists of repeat users who are familiar with the road and its characteristics. Such roads are often unpaved.

Consideration of the unique vehicle types that use agricultural access roads is important in their design. For purposes of these guidelines, rural agricultural access roads consist of roads that are used regularly or seasonally for access to farms by agricultural equipment, such as combines, that are wider than a typical 8.5-ft [2.6-m] truck. Roads that provide frequent access to farms for conventional trucks, but not for wider equipment, should be treated as rural commercial or industrial access roads. Roads that provide access to farms but are used only occasionally by conventional trucks and are not used by wider equipment, should be treated as either rural major access or rural minor access roads depending upon the function and characteristics of the road.

2.3.5 Rural Recreational and Scenic Roads

Recreational and scenic roads serve specialized land uses, including parks, tourist attractions, and recreational facilities, such as campsites or boat-launch ramps, and are found primarily in rural areas. Traffic is open to the general public, and their users are more likely than users of other functional subclasses of local roads to consist of unfamiliar drivers. Recreational and scenic roads do not generally carry significant volumes of truck traffic, but do serve recreational vehicles including motor homes, campers, and passenger cars pulling boats and other trailers. In many cases, these roads may carry highly seasonal traffic volumes. Recreational and scenic roads may accommodate a wide range of speeds and trip lengths may be fairly long. Such roads can be either paved or unpaved. Additional design guidance for this category of roadway is available in the U.S. Forest Service *Road Preconstruction Handbook* (**17**).

2.3.6 Rural Resource Recovery Roads

Resource recovery roads are local roads serving logging, mining, oilfield, or similar operations. Such roads are typically found only in rural areas, and frequently in very remote areas. Resource recovery roads are distinctly different from the other functional subclasses of low-volume roads in that they are used primarily by vehicles involved with the resource recovery activities and the driving population consists primarily or exclusively of professional drivers with large vehicles. In some cases, traffic operations on resource recovery roads are enhanced through radio communication between drivers, enabling such roads to be built and to operate as single-lane roads. Most resource recovery roads are unpaved. The design guidelines for this category of road take into account the remoteness of their setting, the types of vehicles that generally use such a road, and the professional experience of the drivers of those vehicles. Additional design guidance for this category of roadway is available in the U.S. Forest Service *Road Preconstruction Handbook* (**17**).

2.3.7 Urban Major Access Streets

Urban major access streets, like major access roads in rural areas, serve a dual function of providing access to adjacent property as well as providing through or connecting service between other local roads or higher type facilities. Urban major access roads are generally shorter than major access roads in rural areas, but their function in serving more through traffic than most local roads is much the same. Thus, urban major access streets that are functionally classified as local roads often approach the status of a minor collector. Minor collector streets in urban and suburban areas should be treated as urban major access streets for purposes of the guidelines. Any specific role assigned to particular urban major access streets in local pedestrian and bicycle planning should be considered in their design.

2.3.8 Urban Residential Streets

Urban residential streets typically serve to provide access to single- and multiple-family residences in urban areas. Motorists, pedestrians, and bicyclists using such streets generally include only residents and their visitors. Use of such streets by large trucks and other heavy vehicles is rare, except for occasional use by delivery and maintenance vehicles. Accessibility for school buses, fire trucks, other emergency vehicles, and maintenance vehicles such as snow plows and garbage trucks is an important consideration in the design of residential streets.

2.3.9 Urban Industrial or Commercial Access Streets

Urban industrial or commercial access streets, like their rural counterparts, serve development that may generate a substantial volume of trucks or other heavy vehicles. The primary function of such a street is typically to provide access from a factory or another industrial or commercial site to the local or regional highway network. Pedestrian and bicycle usage may be expected on some industrial or commercial access streets. Industrial or commercial access streets are typically quite short, can be paved or unpaved, and may or may not carry traffic from smaller streets. The main defining characteristic of an industrial or commercial street is that its design is influenced by the heavy vehicles using the street.

2.3.10 Other Urban Facilities

Urban agricultural access roads, recreational and scenic roads, and resource recovery roads are rare, but where they occur, they should be designed like their rural counterparts.

2.3.11 Roads that Meet the Definition of More than One Functional Subclass

Some roads meet the definition of more than one of the functional subclasses defined above. For example, a given road might be considered both a rural minor access road and a rural agricultural access road. Another road might be considered both a rural major access road and a recreational and scenic road. In such cases, the road should be evaluated using the design guidelines applicable to each functional class, as presented in Chapter 4, and the higher of the applicable design guidelines should be applied.

2.4 DESIGN SPEED OR OPERATING SPEED

Speed has always been a primary defining variable in the development and presentation of geometric design criteria. Current AASHTO policy specifies design criteria in increments of 5 mph [10 km/h]. Designers select a design speed which is appropriate for the roadway and that speed is used to correlate the various features of the design. The selected design speed should be a logical one with respect to the anticipated operating speed, topography, adjacent land use, and functional classification of the road. In selection of design speed, every effort should be made to attain a desired level of safety, mobility, and efficiency within the constraints of environmental quality, economics, aesthetics, and social or political impacts. The selected design speed should be consistent with the speeds that drivers are likely to expect on a given road. Where a reason for limiting speed is obvious to approaching drivers or bicyclists, they are more apt to accept lower speed operation than where there is no apparent reason.

One of the design guidelines presented in Chapter 4 varies as a function of speed, as follows:

- Low speed—0 to 45 mph [0 to 70 km/h], and
- High speed—more than 45 mph [70 km/h].

2.5 TRAFFIC VOLUMES

The projected average daily traffic volume (ADT) should be used as the basis for design. Usually, the year for which traffic is projected is 20 years from the date of completion of construction, but may range from the current year to more than 20 years in the future depending upon the nature of the improvement. Where traffic volumes vary substantially from season to season, design should be based on the ADT during the peak season. Traffic volume growth rates on low-volume roads are generally modest, and some roads may experience future traffic volume decreases. However, the designer should be alert to the possibility of future development that might affect traffic volume growth, especially in or near urban areas. If new development that would increase the design volume above 2,000 vehicles per day is anticipated on a local or minor collector road within the period for which traffic volumes are projected, then Chapter 5 or 6 of the AASHTO Green Book (**5**) should be used instead of the design guidelines presented here. Where future development is uncertain, a project with a projected volume of 2,000 vehicles per day or less may be designed in accordance with the design guidelines presented in Chapter 4, but the basis for this decision should be documented.

For some design criteria for new construction in Chapter 4, traffic volumes on low-volume roads (2,000 vehicles per day or less) are stratified into several levels. The volume ranges used for specific new construction design criteria may include:

- 100 vehicles per day or less,
- 101 to 250 vehicles per day,
- 251 to 400 vehicles per day, or
- 401 to 2,000 vehicles per day.

3 Rationale for Development of Design Guidelines

3.1 INTRODUCTION

This chapter explains the rationale for and approach to development of the design guidelines presented in Chapter 4. The rationale includes a discussion of the unique characteristics of low-volume roads, a discussion of the basis for the design guidelines, an explanation of the risk-assessment approach used to develop the guidelines, an explanation of the differences between the guidelines for new construction and for improvement of existing roads, and a discussion of the need for flexibility in applying the design guidelines.

3.2 UNIQUE CHARACTERISTICS OF LOW-VOLUME ROADS

The design guidelines presented in Chapter 4 are based on the unique characteristics of low-volume roads. The fundamental characteristics of low-volume roads that distinguish them from other types of roads are:

- The traffic volumes of such roads are, by definition, low. All low-volume roads have average daily traffic volumes of 2,000 vehicles per day or less, and many such roads have volumes that are much less than the 2,000-vehicle-per-day threshold value. These low traffic volumes mean that encounters between vehicles that represent opportunities for crashes to occur are rare events and that multiple-vehicle collisions of any kind are extremely rare events.
- The limitation of the scope of these guidelines to local and minor collector roads means that many motorists using the road have traveled it before and are familiar with its features. Geometric design features that might surprise an unfamiliar driver will be anticipated by the familiar driver.

Because of these unique characteristics, design guidelines for low-volume roads can be less stringent than those used for higher volume roads or roads that serve primarily unfamiliar drivers. The functional subclasses of low-volume roads presented in Chapter 2 permit the design guidelines to vary with the expected proportion of unfamiliar drivers. Similarly, design guidelines for low-volume roads also vary with the expected design traffic volume level.

3.3 BASIS FOR DESIGN RECOMMENDATIONS

Design criteria for streets, roads, and highways are based on a wide range of considerations. Operational efficiency, low crash frequency and severity, constructability, and maintainability are of primary importance. While limiting crash frequency and severity is fundamentally the most important factor in design criteria, the other considerations play a meaningful role as well. An overriding concern in development of design criteria is the concept of flexibility to accommodate future uncertainty. A well-designed road should reflect the potential for changes in traffic volumes, patterns, and operating conditions. Similarly, drivers with a wide range of skills, operating a wide variety of vehicles, may use a highway, including unfamiliar or less skilled drivers, a combination of passenger cars, trucks, and other vehicle types, and nonmotorized users including pedestrians and bicyclists. Road designs should be developed to be appropriate for the specific context of each facility including rural areas, rural towns, suburban areas, urban areas, and urban core areas. Low-volume roads can be found in each of these contexts, with the possible exception of urban core areas.

It is important to understand how design criteria fit within the overall design process. Design criteria are generally employed as minimum or limiting values, beyond which the designer should not go unless unusual circumstances create a site-specific need. Design criteria typically express geometric dimensions in terms of minimum values (lane width, shoulder width, curve radius, stopping sight distance) or maximum values (grades). Design criteria, as published and used, thus tend to direct or limit a design's basic characteristics; and the intent of such criteria is that they be followed with relatively few exceptions.

Design criteria to reflect the considerations described above have been developed to have minimal effect on crash frequency and severity. In other words, design for basic geometric elements such as alignment and cross section have been historically derived to operate effectively across the wide range of conditions that might occur across a highway system. Past design criteria have not typically been based on a strict or rigorous cost-effectiveness approach, but have incorporated values that are judged to be reasonable and prudent given the overall costs, impacts, and benefits to be derived systemwide from the highway system.

The design guidelines for low-volume roads presented in this document are based on a risk assessment performed by Neuman (**11**). This risk assessment was intended to establish design criteria for low-volume roads that, when applied systemwide, will operate comparably to those presented in the AASHTO Green Book (**5**) for higher volume roads. However, because of the unique characteristics of low-volume roads discussed earlier in this chapter, appropriate design criteria for such roads differ from those for higher volume roads. The analysis approach used in this risk assessment is presented below. Other research and sources of information consulted in the preparation of these guidelines have included existing AASHTO policies [the Green Book (**5**), the *Roadside Design Guide* (**3**), the Pedestrian Guide (**1**), and the Bicycle Guide (**4**)], TRB Special Report 214 (**14**), NCHRP Report 362 (**20**), NCHRP Report 383 (**9**), NCHRP Report 400 (**7**), horizontal curve research by Zegeer et al. (**19**), guardrail research by Stephens (**12**) and by Wolford and Sicking (**18**), design guidelines developed by the U. S. Forest Service (**17**) and the Transportation Association of Canada (**13**), and the *Neighborhood Street Design Guidelines* (**10**) developed by the Institute of Transportation Engineers (ITE).

An important component of the design guidelines for low-volume roads is the incorporation of substantial design flexibility based on the exercise of judgment by qualified engineering professionals who are familiar with site conditions and local experience. The important role of design flexibility in the guidelines is addressed later in this chapter.

3.4 DEVELOPMENT OF DESIGN GUIDELINES THROUGH RISK ASSESSMENT

The risk assessment by Neuman (**11**) recommends that design criteria for low-volume roads should be based on tradeoffs between two factors:

- demonstrable differences in construction and maintenance costs, and
- estimated impacts on traffic crash frequency or severity.

This approach highlights crash frequency and severity and cost (and hence, cost-effectiveness in a more direct sense) as the only appropriate basis for defining minimum design criteria or values for these unique facilities. Other factors such as level of service, travel time savings, and driver comfort and convenience are not considered of sufficient importance for low-volume roads to influence their fundamental design criteria.

Because it is derived from a formal risk assessment, the design philosophy recommended for low-volume roads is based fundamentally on limiting crash frequency and severity. Moreover, the philosophy focuses on direct comparison of known or expected crash reduction benefits and system costs. This tradeoff implies that public funds spent to improve such roads in the name of crash reduction should be spent only where there is likely to be an actual crash reduction benefit in return. This, in turn, assures that highway funds expended for crash reduction purposes on all highways (not just low-volume roads) will be available for use where they are most needed (i.e., where meaningful crash reduction benefits can reasonably be expected).

3.4.1 Risk Assessment Approach

The risk assessment by Neuman (**11**) addressed roads with average daily traffic volumes of 400 vehicles per day or less and represents a comparison between crash risk for low-volume roads designed in accordance with the guidelines presented in Chapter 4 of this document and roads designed in accordance with Chapter 5 of the AASHTO Green Book (**5**). The guidelines concerning threshold or acceptable risk levels for new construction used by Neuman (**11**) were:

- For urban or low-speed facilities, an acceptable risk is represented by an action or proposed action that is expected to result in no more than one additional traffic crash per kilometer of roadway every 6 to 10 years. This is equivalent to one additional traffic crash per mile of roadway every 4 to 6 years.
- For rural or high-speed facilities, an acceptable risk is represented by an action or proposed action that is expected to result in no more than one additional traffic crash per kilometer of roadway every 10 to 15 years. This is equivalent to one additional traffic crash per mile of roadway every 6 to 9 years.

The risk assessment by Neuman (**11**) did not address new construction of roads in the range of average daily traffic volumes from 401 to 2,000 vehicles per day. Therefore, the design guidance for new construction of such roads presented in Chapter 4 of this document is essentially equivalent to the guidance in Chapter 5 of the AASHTO Green Book (**5**).

These risk assessment thresholds for rural and urban roadways with average daily traffic volumes of 400 vehicles per day or less are consistent with those used to evaluate roadway widths in NCHRP Report 362 (**20**), which was the basis for the current lane and shoulder width design values for rural highways in the AASHTO Green Book (**5**). Although NCHRP Report 362 considers roadways with higher ADTs than those addressed in these guidelines, it provided a model for the risk assessment of low-volume roads.

The acceptable risk levels represented by the thresholds presented above were applied by Neuman (**11**) in the research to develop the design guidelines presented in Chapter 4. Determination of expected risk levels was based on a synthesis of the best available research on the quantitative relationships between key geometric design elements and the frequency and severity of crashes.

The threshold or acceptable risk levels given above represent maximum risk levels over an extensive roadway system consisting of many sites; these maximum risk levels are not likely to occur everywhere, but only at sites where minimum or controlling geometry is incorporated in a design. The threshold risk levels presented above were used in the research that developed the guidelines; they are not intended for use in the assessment of individual sites.

As an example of the risk assessment approach, the use of a minimum radius curve in a project designed in accordance with Chapter 4 of these guidelines may result in a crash rate slightly higher than a curve designed to the minimum radius shown in the AASHTO Green Book (**5**), but the crash frequency and severity of the rest of the roadway, consisting of tangents and larger radius curves, should be unaffected. Consider a horizontal curve with a design speed of 60 mph [100 km/h] and a maximum superelevation rate of 6 percent on a rural major access road with an ADT of 400 vehicles per day. The minimum radius for such a curve designed in accordance with the AASHTO Green Book (**5**), is 1,330 ft [437 m]. Neuman (**11**) determined that the difference in crash frequency and severity between a curve with a radius of 1,330 ft [437 m] and a curve with a radius of 830 ft [250 m] for the specified conditions would be less than 1.6 crashes per mi [1.0 crashes per kilometer] over a period of 10 years. Therefore, the use of a minimum radius of 830 ft [250 m] can be recommended for horizontal curves on low-volume roads under the specified conditions. All of the design guidelines presented in Chapter 4 are based on risk analyses of this type conducted by Neuman (**11**).

This example does not imply that the minimum radius of 830 ft [250 m] is appropriate for all horizontal curves on low-volume roads, any more than the minimum radius of 1,330 ft [437 m] is appropriate for all horizontal curves on higher volume roads. Rather, the design guidelines in Chapter 4 give the designer flexibility to use radii as small as 830 ft [250 m] should site-specific conditions warrant this. The guidelines were developed with the understanding that a designer is expected to exercise engineering judgment in selecting any design value, whether it is a minimum value or not.

For existing roads, application of the design guidelines in Chapter 4 should result in a slight reduction in systemwide crash frequency and severity. The safety performance of most existing low-volume roads will be unaffected by the guidelines, but improvements that should have a positive effect on reducing crash frequency and severity are recommended at locations on existing roads where site-specific crash patterns are found.

3.4.2 Expected Systemwide Effects on Crash Frequency and Severity

Overall, the net effect on systemwide safety of applying these design guidelines for low-volume roads will undoubtedly be small. Any higher expected crash frequencies that might occur would take place at limited sites, not over the entire length of the road system. For example, not every horizontal curve would be affected, only those curves that are designed to minimum radii. Similarly, only a portion of the length of any roadway is likely to lack a clear zone. Thus, it is highly unlikely that, even on roadways with several design elements at minimum values, the risk thresholds presented above would be exceeded. Furthermore, for projects on existing roads there should be a net reduction in crash frequency and severity because existing roads with site-specific crash patterns will be improved. Thus, the net effect of applying these design guidelines systemwide for low-volume roads should be a change in crash frequency and severity that is so small as to be negligible.

The use of risk assessment as a basis for the design guidelines is intended to focus public spending for low-volume roads on improvements at locations where it can be expected to provide substantial crash reduction benefits and to discourage spending at locations where little or no crash reduction benefit would be expected. This will allow scarce public funds to construct and repair more facilities rather than spending major amounts of funds in one location while not addressing other locations in need.

The AASHTO *Highway Safety Manual* (HSM) (**2**) provides guidance on highway safety management and crash prediction methods for specific roadway types. HSM methods confirm that low-volume roads are expected to experience fewer crashes than roads with traffic volumes above 2,000 vehicles per day. HSM techniques can be applied to analyze highway improvement needs of specific roads based on their observed crash history. Highway agencies have also begun using systemic approaches to highway improvement, which may include making improvements to existing roads based on the potential for future crashes, even if no crashes have yet occurred. However, there is very little potential for crash reduction to result from systemic improvements to low-volume roads in the absence of specific crash patterns. The most effective systemwide results will be obtained if systemic improvements are focused on roads with traffic volumes above 2,000 vehicles per day and the improvement strategies presented in these guidelines are applied to existing low-volume roads.

3.5 GUIDELINES FOR NEW CONSTRUCTION VERSUS IMPROVEMENT OF EXISTING ROADS

Separate design guidelines are presented in Chapter 4 for new construction and improvement of existing roads. In most cases, specific design criteria are presented for new construction of low-volume roads. For roads with design volumes of 400 vehicles per day or less, these design criteria are generally less restrictive than those used in new construction of higher volume roads such as those discussed in the AASHTO Green Book (**5**).

Projects on existing low-volume roads may involve reconstruction, resurfacing, rehabilitation, restoration, or other types of improvements. Changes to roadway or roadside geometrics during such projects are generally recommended only where there is a documentable site-specific crash pattern that can potentially be corrected by a roadway or roadside improvement. Thus, this guide implements a performance-based design approach for improvement of existing roads. Where documentable site-specific crash patterns do not exist, it is unlikely that any roadway or roadside improvement would provide substantial crash reduction benefits. The design guidelines in Chapter 4 provide advice to the designer on specific situations in which geometric improvements may be desirable on existing roads.

There are a wide variety of sources that may be considered in investigating and documenting the existence of site-specific crash risk. These naturally include crash history data. Because low-volume roads have few crashes, a long time period, typically 5 to 10 years, should be considered in reviewing crash patterns. However, even when 5 to 10 years of crash data are available, these data will often be so sparse that other indicators of crash risk should be considered, as well. Such other indicators may include field reviews to note skid marks or roadside damage, speed data (which may indicate whether speeds are substantially higher than the intended design speed), or concerns raised by police or local residents. These indicators should be fully considered in the assessment of site-specific crash risk because, as stated above, assessments of low-volume roads should not usually be based on crash data alone.

In projects on existing roads, consideration should also be given to maintaining consistency in geometric design features and consistency in speed between adjacent sections of roadway.

3.6 DESIGN FLEXIBILITY

The design guidelines in Chapter 4 are intended to provide great flexibility for the designer to exercise engineering judgment about the appropriate geometric and roadside designs for specific projects. Even for new construction projects, where specific design criteria are recommended in Chapter 4, the guidelines provide flexibility for the designer to change those criteria for specific projects where such changes appear appropriate. The designer has the flexibility to use reduced design criteria, where judgment indicates that this can be accomplished without substantially affecting crash risk, or to increase the design criteria to the levels used for higher volume roads as discussed in the AASHTO Green Book (**5**).

Even more flexibility than for new construction projects is provided to the designer for projects on existing roads, because the guidelines in Chapter 4 do not include quantitative design criteria for such projects. Rather, the designer is discouraged at most sites from making unnecessary geometric design and roadside improvements, but is encouraged to look for evidence of site-specific crash patterns and to focus expenditures on those sites where a site-specific crash pattern exists that is potentially correctable by a specific roadway or roadside improvement.

Where designers exercise judgment and develop a project using design criteria that differ from those presented in the Chapter 4 guidelines, or where a site-specific crash pattern is identified and used as the basis for a design decision, the designer should document the decision making process in writing. This is not intended to imply that a formal design exception is needed; however, it is good practice to document key project decisions in writing.

The guidelines encourage the designer to exercise engineering judgment based on a thorough knowledge of the principles of highway design, traffic engineering, highway safety engineering, and specific knowledge of local conditions. Thus, the flexibility provided by these guidelines is intended to be exercised by a qualified engineer.

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4 Design Guidelines

4.1 INTRODUCTION

This chapter presents design guidelines for specific aspects of the design of low-volume roads including cross section (traveled way and shoulder widths), horizontal alignment, stopping sight distance, intersection sight distance, roadside design, pedestrian and bicycle facilities, unpaved roads, and two-way single-lane roads.

The design guidelines for new construction and projects on existing roads presented in this chapter apply to low-volume local and minor collector roads with design traffic volumes of 2,000 vehicles per day or less. New construction for roads with design traffic volumes over 2,000 vehicles per day should generally be designed in accordance with AASHTO Green Book (**5**) criteria. However, design flexibility for roads with traffic volumes of 2,000 vehicles per day or less should be especially encouraged.

4.2 CROSS SECTION

The key elements of cross section design for a roadway are traveled way width and, in rural areas, shoulder width. Cross section design criteria for lower volume roads generally address total roadway width (traveled way plus shoulders) rather than having separate criteria for lane and shoulder width. Many lower volume roadways have no painted edgelines and do not have paved shoulders of a material that contrasts with the traveled-way pavement, so there may be no clear demarcation between the traveled way and shoulders. Design guidelines for cross sections of paved two-lane roads in new construction projects and on existing low-volume roads are presented in Sections 4.2.1.1 and 4.2.1.2, respectively. Separate discussions of unpaved roads and single-lane roads are provided in Sections 4.9 and 4.10, respectively.

4.2.1 New Construction

The design guidelines for cross section in new construction projects on low-volume roads differ between rural and urban areas. Each set of design guidelines is presented below. While the quantitative design guidelines for new construction address only total roadway widths, designers should also give consideration to the appropriate right-of-way width. In new construction projects, ample right-of-way should be obtained, whenever practical, to accommodate possible future widening of the roadway.

4.2.1.1 Low-Volume Roads in Rural Areas

Table 4-1 presents the guidelines for total roadway widths for newly constructed roads in rural areas. Total roadway width is the sum of the traveled way width and the usable shoulder width. Usable shoulder width includes graded shoulders, but not roadside slopes. The total roadway width criteria vary from 18 to 26 ft [5.4 to 8.0 m] with the functional subclass and the design speed of the road. The roadway width values for roads with design volumes of 400 vehicles per day or less were developed in research by Neuman (**11**) from several sources. The primary source for roadway widths was NCHRP Report 362 (**20**); other sources included TRB Special Report 214 (**14**), the U. S. Forest Service (USFS) (**17**), and the Transportation Association of Canada (**13**). The roadway widths for major access roads with design volumes greater than 400 vehicles per day are based on Chapter 5 of the AASHTO Green Book (**5**). For application of Table 4-1, all low-volume roads with design volumes greater than 400 vehicles per day should be treated as major access roads.

The roadway width guidelines for major access roads, minor access roads, and recreational and scenic roads are based primarily on travel by passenger cars and recreational vehicles. Widths for industrial or commercial access roads, resource recovery roads, and agricultural roads consider more frequent use by larger trucks and, in the case of agricultural access roads, use by wide agricultural equipment. These greater widths for industrial or commercial access roads, resource recovery roads, and agricultural access roads reflect the offtracking and maneuverability needs and the greater widths of the larger vehicles using these roads. The ability of vehicles in opposing directions of travel to pass one another is an important design consideration for rural roads. Access past parked vehicles is not a major concern because parking on rural roads is not common. The increased cross section widths for industrial or commercial access roads, resource recovery roads, and agricultural access roads should not be construed as based on a safety need. It should be noted that the roadway widths for agricultural access roads are applicable on roads used by agricultural equipment wider than a typical 8.5-ft [2.6-m] truck. Some low-volume rural roads, particularly in recreational and scenic areas, may serve substantial bicycle volumes; designers may consider adjusting the roadway widths in Table 4-1 in such cases.

The choice of the appropriate functional subclass is key to determining the appropriate roadway width. Where minimum roadway widths are used for a selected functional subclass, the designer should consider providing a wider roadway at sharp horizontal curves. By contrast, widths less than the minimums shown in Table 4-1 may be appropriate adjacent to historic structures or in other constrained locations. In determining appropriate roadway widths, the designer should refer to the discussion of design flexibility in Section 3.6.

Table 4-1. Guidelines for Total Roadway Width for New Construction of Low-Volume Roads in Rural Areas

U.S. Customary

Design Speed (mph)	Major Access Road by Design Volume Level (veh/day): 400 or Less	Major Access Road by Design Volume Level (veh/day): 401 to 2,000	Minor Access Road	Recreational and Scenic Road	Industrial/ Commercial Access Road	Resource Recovery Road	Agricultural Access Road
			Total Roadway Width (ft) by Functional Subclass[1]				
15	18.0	23.0[2]	18.0	18.0	20.0	20.0	22.0
20	18.0	23.0[2]	18.0	18.0	20.0	20.0	24.0
25	18.0	23.0[2]	18.0	18.0	21.0	21.0	24.0
30	18.0	23.0[2]	18.0	18.0	22.5	22.5	24.0
35	18.0	23.0[2]	18.0	18.0	22.5	22.5	24.0
40	18.0	23.0[2]	18.0	20.0	22.5	—	24.0
45	20.0	25.0	20.0	20.0	23.0	—	26.0
50	20.0	25.0	20.0	20.0	24.5	—	—
55	22.0	25.0	—	22.0	—	—	—
60	22.0	25.0	—	—	—	—	—

Metric

Design Speed (km/h)	Major Access Road by Design Volume Level (veh/day): 400 or Less	Major Access Road by Design Volume Level (veh/day): 401 to 2,000	Minor Access Road	Recreational and Scenic Road	Industrial/ Commercial Access Road	Resource Recovery Road	Agricultural Access Road
			Total Roadway Width (m) by Functional Subclass[1]				
20	5.4	7.0[2]	5.4	5.4	6.0	6.0	6.6
30	5.4	7.0[2]	5.4	5.4	6.0	6.0	7.2
40	5.4	7.0[2]	5.4	5.4	6.4	6.4	7.2
50	5.4	7.0[2]	5.4	5.4	6.8	6.8	7.2
60	5.4	7.0[2]	5.4	5.4	6.8	6.8	7.2
70	6.0	7.6	6.0	6.0	7.0	—	8.0
80	6.0	7.6	6.0	6.0	7.4	—	—
90	6.6	7.6	—	6.6	—	—	—
100	6.6	7.6	—	—	—	—	—

Note: Total roadway width includes the width of both traveled way and usable shoulders.
[1] All low-volume roads with design volumes greater than 400 veh/day should be treated as major access roads
[2] For roads in mountainous terrain with design volumes up to 600 veh/day, use 20.0-ft [6.0-m] total roadway width.

Small differences in the existing or proposed dimensions from those shown in Table 4-1 may be completely acceptable. For example, on roads used by trucks or wider agricultural equipment, designers should have the discretion to consider the actual widths of vehicles expected to use a particular road and modify the width guidelines in Table 4-1 accordingly.

Where pedestrian facilities are provided, they must be accessible to and usable by individuals with disabilities (**6, 16**). Additional design guidance for pedestrian facilities may be found in local policies; the AASHTO *Guide for Planning, Operation, and Design of Pedestrian Facilities* (**1**); and the *Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way* (**15**).

4.2.1.2 Low-Volume Roads in Urban Areas

As in rural areas, the cross-section width guidelines for low-volume roads in urban areas are related to basic operational needs. Speeds are lower, trip lengths and lengths of low-volume roads are generally much shorter, and available right-of-way width is much less than in rural areas. The major functional needs for low-volume roads in urban areas include the ability for vehicles in opposite directions to pass one another, the need for vehicles to pass parked or stopped vehicles, the need to provide access for fire trucks and other emergency vehicles, and the need to accommodate occasional larger delivery vehicles.

Cross-section widths for urban major access roads and urban industrial or commercial access roads should generally be the same as those shown for comparable rural roads in Table 4-1. Greater widths should be provided where parking is permitted.

Cross section width guidelines for urban residential streets are shown in Table 4-2. These widths incorporate consideration of access for fire trucks and other emergency vehicles. Parking should usually be permitted on both sides of residential streets. Reduced widths may be appropriate where parking is restricted to one side of the street. The guidelines in Table 4-2 are adapted from the ITE *Neighborhood Street Design Guidelines* (**10**), which may be consulted for more detailed guidance.

The roadway widths presented in Table 4-2 assume that there is sufficient off-street parking (e.g., driveways and garages) so that on-street parking is used only occasionally by visitors and delivery vehicles.

Where pedestrian facilities are provided, they must be accessible to and usable by individuals with disabilities (**6, 16**). Additional design guidance for pedestrian facilities may be found in local policies; the AASHTO *Guide for Planning, Operation, and Design of Pedestrian Facilities* (**1**); and the *Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right of Way* (**15**).

Table 4-2. Guidelines for Total Roadway Width for New Construction of Two-Lane Urban Residential Streets [adapted from (10)]

Development Density	Dwelling Units per Gross Acre[1]	Number of Channels[2]	U.S. Customary Total Roadway Width (ft) with Parking on: Both Sides	U.S. Customary Total Roadway Width (ft) with Parking on: One Side Only	Metric Total Roadway Width (m) with Parking on: Both Sides	Metric Total Roadway Width (m) with Parking on: One Side Only
Low	≤ 2.0	2	20–22	18	6.1–6.7	5.5
Medium	2.1 to 6.0	3	26–28	24	7.9–8.5	7.3
High	6.1 to 10.0	4	30–32	28	9.2–9.8	8.5
Very high	≥10.1	4	34–38	32	10.4–11.6	9.8

[1] Gross acreage includes land used for the roadway, alleys, pedestrian and bicycle paths, easements, parks, schools, or other neighborhood facilities; use average density for both sides of the street.

[2] Number of channels represents the number of through and parking lanes; for low and medium density development, both the likelihood of parked vehicles and the traffic volume are low, and the occasional parked vehicles may park in a lane designated for through traffic, as needed.

Note: These guidelines should not be used as a substitute for engineering judgment. Specific streets may be wider or narrower than these guidelines based on consideration of site-specific factors.

4.2.2 Existing Roads

The cross-section widths of existing roads need not be modified except in those cases where there is evidence of a site-specific crash pattern. Chapter 3 discusses the types of evidence of a site-specific crash pattern that might be considered. When a site-specific crash pattern that can be mitigated by a wider roadway is identified, the cross section for the portion of the roadway with the identified crash pattern should be widened to at least the total roadway width presented above for new construction.

4.3 BRIDGE WIDTH

The key elements in selecting an appropriate bridge width are the width of the adjacent roadway (traveled way and shoulder widths) and, for existing locations, the crash history of the existing bridge.

4.3.1 New Construction

New bridges are those on new roadways where there is no existing roadway or bridge in place. The widths of new bridges should generally be selected in accordance with the bridge width criteria in the AASHTO Green Book (5), Chapters 5 and 6 for local and minor collector roads, respectively. Those criteria indicate that, for bridges on local roads with ADT of 400 vehicles per day or less, the bridge width should be equal to the width of the traveled way plus 2 ft [0.6 m]. However, when the entire roadway width (traveled way plus shoulders) is paved, the bridge width should be equal to the total roadway width. Bridge width should be measured between the inside faces of the bridge rail or guardrail. Bridges greater than 100 ft [30 m] in length should be evaluated individually to determine the appropriate bridge width. Bridge usage by trucks, recreational vehicles, bicyclists, and pedestrians should also be considered in determining the appropriate width.

One-lane bridges may be provided on single-lane roads and on two-lane roads with design volumes less than 100 vehicles per day where the designer finds that a one-lane bridge can operate effectively. The minimum width of a one-lane bridge should be 15 ft [4.5 m] unless the designer concludes that a narrower bridge can function effectively (e.g., based on the crash history of similar bridges maintained by the same agency). Caution should be exercised in design of one-lane bridges wider than 16 ft [4.9 m] to assure that drivers will not use them as two-lane structures. Simultaneous arrival of two or more opposing vehicles at a one-lane bridge should be rare, given the low traffic volumes, but one-lane bridges should have pull-offs at each end where drivers can wait for traffic on the bridge to clear.

4.3.2 Existing Bridges

Existing bridges can remain in place without widening unless there is evidence of a site-specific crash pattern related to the width of the bridge. As described in Section 3.5, evidence of a site-specific crash pattern may include not only crash history but also other indications such as skid marks, damage to bridge rail or guardrail, and concerns raised by police or local residents. Where an existing bridge needs replacement for structural reasons, but there is no evidence of a site-specific crash pattern, and the existing bridge serves trucks, recreational vehicles, bicyclists, and pedestrians appropriately, the replacement bridge can be constructed with the same width as the existing bridge; this criterion applies to bridges that are reconstructed on the same alignment and bridges that are reconstructed on a more favorable alignment.

4.4 HORIZONTAL ALIGNMENT

For balance in roadway design, all geometric elements should, as far as economically practical, be designed to provide continuous operation at a speed likely to be observed under the general conditions for that roadway. For the most part, this is done through the use of design speed as the overall control. The design of roadway curves is based on understanding the appropriate relation between design speed and curvature and also their joint relations with superelevation and side friction. Although these relations stem from the laws of physics, the actual values for use in design depend on practical limits and factors determined more or less empirically over the range of variables involved.

A key parameter that represents the friction demand for a vehicle traversing a horizontal curve is the side friction factor, which can be estimated as:

U.S. Customary	Metric
$$f = \frac{V^2}{15R} - 0.01e$$	$$f = \frac{V^2}{127R} - 0.01e \qquad (4\text{-}1)$$
where: f = side friction factor; V = vehicle speed, mph; R = radius of curve, ft; and e = rate of roadway superelevation, percent.	where: f = side friction factor; V = vehicle speed, km/h; R = radius of curve, m; and e = rate of roadway superelevation, percent.

A fundamental objective in horizontal curve design is to select a radius of curve, R, such that the side friction factor, f, of a vehicle traversing the curve at the design speed does not exceed a specified threshold value. To achieve this, Equation 4-1 can be recast as:

U.S. Customary	Metric
$$R_{min} = \frac{V^2}{15(0.01e_{max} + f_{max})}$$	$$R_{min} = \frac{V^2}{127(0.01e_{max} + f_{max})} \qquad (4\text{-}2)$$
where: R_{min} = minimum curve radius, ft; e_{max} = maximum rate of superelevation permitted by highway agency policy; and f_{max} = maximum side friction factor.	where: R_{min} = minimum curve radius, m; e_{max} = maximum rate of superelevation permitted by highway agency policy; and f_{max} = maximum side friction factor.

The values of f_{max} and R_{min} used in design of higher volume roads (i.e., roads with design volumes greater than 2,000 vehicles per day) are specified in Chapter 3 of the AASHTO Green Book (**5**) and are presented here in Table 4-3. Maximum superelevation rates from 4 to 12 percent may be used in the design of such curves. Guidance in selection of an appropriate maximum superelevation rate is provided by the AASHTO Green Book (**5**). The values of f_{max} in Table 4-3 are intended to assure the comfort of drivers in traversing a curve. Actual tire–pavement friction data indicate that these criteria provide a substantial margin against loss of control due to skidding on most pavements, even at high speeds.

Table 4-3. Maximum Side Friction Factor and Minimum Radius for Horizontal Curve Design on Higher Volume Roadways (Design Volume > 2000 veh/day) (5)

U.S. Customary

		Minimum Radius (ft), R_{min}				
		Max. Superelevation Rate (%), e_{max}				
Design Speed (mph)	Maximum Design Side Friction Factor, f_{max}	4	6	8	10	12
10	0.38	25	25	25	25	25
15	0.32	40	40	40	35	35
20	0.27	85	80	75	70	70
25	0.23	155	145	135	125	120
30	0.20	250	230	215	200	190
35	0.18	370	340	315	290	270
40	0.16	535	485	445	410	380
45	0.15	710	645	585	540	500
50	0.14	925	835	760	695	640
55	0.13	1185	1060	960	875	805
60	0.12	1500	1335	1200	1090	1000

Metric

		Minimum Radius (m), R_{min}				
		Max. Superelevation Rate (%), e_{max}				
Design Speed (km/h)	Maximum Design Side Friction Factor, f_{max}	4	6	8	10	12
15	0.40	10	10	10	10	10
20	0.35	10	10	10	10	10
30	0.28	20	20	20	20	20
40	0.23	45	45	40	40	35
50	0.19	85	80	75	70	65
60	0.17	135	125	115	105	100
70	0.15	205	185	170	155	144
80	0.14	280	250	230	210	195
90	0.13	375	335	305	275	255
100	0.12	490	435	395	360	330

A risk assessment by Neuman (**11**) found that because established horizontal curve design criteria are based on driver comfort levels, rather than loss of control, the criteria for f_{max} and R_{min} can be relaxed for low-volume roads with average daily traffic volumes of 400 vehicles per day or less with no discernable change in crash risk. The specific criteria applicable to horizontal curve design for new construction projects and for existing low-volume roads are presented in Sections 4.4.1 and 4.4.2, respectively.

4.4.1 New Construction

The following guidelines are recommended for design of horizontal curves in new construction of low-volume roads:

- For the design of low-volume roads with design volumes of 400 vehicles per day or less without substantial truck and recreational vehicle volumes, acceptable operations can be obtained with smaller curve radii than those shown in Table 4-3. Design radii based on a reduction in design speed of 5 to 10 mph or 10 to 20 km/h, may be used. The maximum reduction in design speed of 10 mph or 20 km/h is generally appropriate for roadways with speeds of 45 mph or 70 km/h or more and with design volumes of 250 vehicles per day or less. For roadways with design volumes of 250 to 400 vehicles per day without substantial truck volumes, the appropriate maximum reduction in design speed is 10 mph or 15 km/h.
- For the design of low-volume roads with design volumes of 400 vehicles per day or less that carry substantial recreational vehicle or truck traffic, design radii based on no reduction in design speed should be used at very low speeds (e.g., 15 mph or 20 km/h). This guideline reflects the greater likelihood of truck rollover at low speeds. At higher speeds, design radii based on a reduction in speed of no more than 5 mph or 10 km/h, may be used.

The specific guidelines for the design of horizontal curves in new construction projects are presented separately for seven categories of low-volume roads. These are:

- rural major access, minor access, and recreational and scenic roads with design volumes of 250 vehicles per day or less;
- rural major access, minor access, and recreational and scenic roads with design volumes from 251 to 400 vehicles per day;
- rural industrial or commercial access, agricultural access, and resource recovery roads with design volumes up to 400 vehicles per day;
- urban major access streets with design volumes of 250 vehicles per day or less and urban residential streets;
- urban major access streets with design volumes from 251 to 400 vehicles per day;
- urban industrial or commercial access streets with design volumes up to 400 vehicles per day; and
- low-volume roads of any functional subclass with design volumes of 401 to 2,000 vehicles per day.

Horizontal curve design criteria for new construction of roads in each of these seven categories are presented below.

4.4.1.1 Rural Major Access, Minor Access, and Recreational and Scenic Roads (250 Vehicles per Day or Less)

The design of horizontal curves for major access, minor access, and recreational and scenic roads in rural areas is based on the expectation that the proportion of large trucks in these functional subclasses is relatively low. Newly constructed rural roads in these subclasses should be designed using the limiting values of f_{max} and R_{min} shown in Table 4-3, whenever practical. In constrained situations, for roads with design volumes of 250 vehicles per day or less, horizontal curves may be designed using the limiting values for f_{max} and R_{min} presented in Table 4-4. This table incorporates reductions in design speed up to 10 mph or 20 km/h based on the design principles presented above. Table 4-4 is appropriate in constrained situations, where providing a horizontal curve designed in accordance with Table 4-3 would result in significant additional costs for earthwork or right-of-way acquisition or would have significantly greater environmental impacts. Design superelevation and superelevation transitions for this category of low-volume roads is discussed in Section 4.4.1.8.

4.4.1.2 Rural Major Access, Minor Access, and Recreational and Scenic Roads (251 to 400 Vehicles per Day)

As in the previous category, rural major access, minor access, and recreational and scenic roads with design volumes from 251 to 400 vehicles per day should be designed with horizontal curves based on the limiting values of f_{max} and R_{min} shown in Table 4-3, whenever practical. In constrained situations, the limiting values of f_{max} and R_{min} shown in Table 4-5 may be used. Table 4-5 incorporates reductions in design speed up to 10 mph or 15 km/h based on the design principles presented above. Design of superelevation and superelevation transitions for this category of very low-volume roads is discussed in Section 4.4.1.8.

4.4.1.3 Rural Industrial or Commercial Access, Agricultural Access, and Resource Recovery Roads (400 Vehicles per Day or Less)

Horizontal curves on rural industrial or commercial access, agricultural access, and resource recovery roads with design volumes of 400 vehicles per day or less should be designed using the limiting values of f_{max} and R_{min} shown in Table 4-3, whenever practical. In constrained situations, the limiting values of f_{max} and R_{min} shown in Table 4-6 may be used. Table 4-6 incorporates reductions in design speed up to 5 mph or 10 km/h. Lower reductions in design speed are used for industrial or commercial, agricultural access, and resource recovery roads because these functional subclasses are more likely than other subclasses to carry substantial proportions of large trucks. Design of superelevation and superelevation transitions for this category of low-volume roads is discussed in Section 4.4.1.8.

Table 4-4. Guidelines for Maximum Side Friction Factor and Minimum Radius (New Construction, Design Volume ≤ 250 veh/day, Limited Proportion of Heavy Vehicle Traffic)

U.S. Customary

			Minimum Radius (ft), R_{min}				
			Max. Superelevation Rate (%), e_{max}				
Design Speed (mph)	Reduced Design Speed (mph)	Maximum Design Side Friction Factor, f_{max}	4	6	8	10	12
10	10	0.380	25	25	25	25	25
15	15	0.320	40	40	40	35	35
20	15	0.320	40	40	40	35	35
25	20	0.270	85	80	75	70	70
30	20	0.270	85	80	75	70	70
35	25	0.230	155	145	135	125	120
40	30	0.200	250	230	215	200	190
45	35	0.180	370	340	315	290	270
50	40	0.160	535	485	445	410	380
55	45	0.150	710	645	585	540	500
60	50	0.140	925	835	760	695	640

Metric

			Minimum Radius (m), R_{min}				
			Max. Superelevation Rate (%), e_{max}				
Design Speed (km/h)	Reduced Design Speed (km/h)	Maximum Design Side Friction Factor, f_{max}	4	6	8	10	12
15	15	0.400	10	10	10	10	10
20	20	0.350	10	10	10	10	10
30	25	0.315	15	15	10	10	10
40	30	0.280	20	20	20	20	20
50	35	0.255	35	30	30	25	25
60	45	0.210	65	60	55	50	50
70	50	0.190	85	80	75	70	65
80	60	0.170	135	125	115	105	100
90	70	0.150	205	185	170	155	145
100	80	0.140	280	250	230	210	195

Table 4-5. Guidelines for Maximum Side Friction Factor and Minimum Radius (New Construction, Design Volumes from 251 to 400 veh/day, Limited Proportion of Heavy Vehicle Traffic)

U.S. Customary

			Minimum Radius (ft), R_{min}				
			Max. Superelevation Rate (%), e_{max}				
Design Speed (mph)	Reduced Design Speed (mph)	Maximum Design Side Friction Factor, f_{max}	4	6	8	10	12
10	10	0.380	25	25	25	25	25
15	15	0.320	40	40	40	35	35
20	15	0.320	40	40	40	35	35
25	20	0.270	85	80	75	70	70
30	25	0.230	155	145	135	125	120
35	30	0.200	250	230	215	200	190
40	35	0.180	370	340	315	290	270
45	40	0.160	535	485	445	410	380
50	45	0.150	710	645	585	540	500
55	50	0.140	925	835	760	695	640
60	50	0.140	925	835	760	695	640

Metric

			Minimum Radius (m), R_{min}				
			Max. Superelevation Rate (%), e_{max}				
Design Speed (km/h)	Reduced Design Speed (km/h)	Maximum Design Side Friction Factor, f_{max}	4	6	8	10	12
15	15	0.400	10	10	10	10	10
20	20	0.350	10	10	10	10	10
30	25	0.315	15	15	10	10	10
40	30	0.280	20	20	20	20	20
50	40	0.230	45	45	40	40	35
60	50	0.190	85	80	75	70	65
70	60	0.170	135	125	115	105	100
80	65	0.160	165	155	140	130	120
90	75	0.145	240	215	195	180	165
100	85	0.135	325	290	265	240	225

Table 4-6. Guidelines for Maximum Side Friction Factor and Minimum Radius (New Construction, Design Volumes of 400 veh/day or Less, Substantial Proportion of Heavy Vehicle Traffic)

U.S. Customary

Design Speed (mph)	Reduced Design Speed (mph)	Maximum Design Side Friction Factor, f_{max}	Minimum Radius (ft), R_{min} — Max. Superelevation Rate (%), e_{max}: 4	6	8	10	12
10	10	0.380	25	25	25	25	25
15	15	0.320	40	40	40	35	35
20	20	0.270	85	80	75	70	70
25	25	0.230	155	145	135	125	120
30	25	0.230	155	145	135	125	120
35	30	0.200	250	230	215	200	190
40	35	0.180	370	340	315	290	270
45	40	0.160	535	485	445	410	380
50	45	0.150	710	645	590	540	500
55	50	0.140	925	835	760	695	640
60	55	0.130	1190	1060	960	875	805

Metric

Design Speed (km/h)	Reduced Design Speed (km/h)	Maximum Design Side Friction Factor, f_{max}	Minimum Radius (m), R_{min} — Max. Superelevation Rate (%), e_{max}: 4	6	8	10	12
15	15	0.400	10	10	10	10	10
20	20	0.350	10	10	10	10	10
30	30	0.280	20	20	20	20	20
40	40	0.230	45	45	40	40	35
50	45	0.210	65	60	55	50	50
60	55	0.180	110	100	90	85	80
70	65	0.160	165	155	140	130	120
80	70	0.150	205	185	170	155	145
90	80	0.140	280	250	230	210	195
100	90	0.130	375	335	305	275	255

4.4.1.4 Urban Major Access Streets (250 Vehicles per Day or Less) and Urban Residential Streets (400 Vehicles per Day or less)

Horizontal curves on urban major access streets with design volumes of 250 vehicles per day or less and on urban residential streets with design volumes of 400 vehicles per day or less should be designed in accordance with the limiting values of f_{max} and R_{min} presented in Table 4-3, whenever practical. In constrained situations, the limiting values of f_{max} and R_{min} shown in Table 4-4 may be used in place of Table 4-3. Design of superelevation and superelevation transitions for this category of low-volume roads is discussed in Section 4.4.1.8.

4.4.1.5 Urban Major Access Streets (251 to 400 Vehicles per Day)

Horizontal curves on urban major access streets with design volumes from 251 to 400 vehicles per day should be designed in accordance with the limiting values of f_{max} and R_{min} presented in Table 4-3, whenever practical. In constrained situations, the limiting values of fmax and R_{min} shown in Table 4-5 may be used in place of Table 4-3. Design of superelevation and superelevation transitions for this category of low-volume roads is discussed in Section 4.4.1.8.

4.4.1.6 Urban Industrial or Commercial Access Streets (400 Vehicles per Day or Less)

Horizontal curves on urban industrial or commercial access streets should be designed in accordance with the limiting values of f_{max} and R_{min} presented in Table 4-3, whenever practical. In constrained situations, the limiting values of f_{max} and R_{min} shown in Table 4-6 may be used in place of Table 4-3. Design of superelevation and superelevation transitions for this category of low-volume roads is discussed in Section 4.4.1.8.

4.4.1.7 Low-Volume Roads of Any Functional Subclass (401 to 2,000 Vehicles per Day)

Horizontal curves on low-volume roads in any functional subclass should be designed in new construction projects in accordance with the limiting values of f_{max} and R_{min} presented in Table 4-3.

4.4.1.8 Superelevation and Superelevation Transitions

Once the radius for a particular horizontal curve has been determined, the selection of the appropriate superelevation and the design of superelevation transitions should proceed in accordance with the criteria presented in Chapter 3 of the AASHTO Green Book (5). Where the horizontal curve design is based on Table 4-3, the superelevation and superelevation transition design should follow the criteria from Chapter 3 of the AASHTO Green Book for the actual roadway design speed. Where the horizontal curve design is based on Tables 4-4, 4-5, or 4-6, the superelevation and superelevation transition design follow the criteria from Chapter 3 of the AASHTO Green Book using the reduced design speed indicated in Tables 4-4, 4-5, or 4-6 in place of the roadway design speed. The criteria in Chapter 3 of the AASHTO Green Book concerning situations where no superelevation is needed apply to low-volume roads based on the roadway design speed or the reduced design speed, as appropriate.

4.4.2 Existing Roads

For improvement projects on existing low-volume roads, the existing horizontal curve geometry should generally be considered acceptable unless there is evidence of a site-specific crash pattern related to horizontal curvature. The following guidelines reflect the results of the risk assessment for horizontal curves on existing roads:

- For curves on low-volume roads with low speeds (design or estimated operating speed of 45 mph [70 km/h] or less), reconstruction without changing the existing curve geometry and cross section is acceptable if the nominal design speed of the curve is within 20 mph or 30 km/h of the design or operating speed, and if there is no clear evidence of a site-specific crash pattern associated with the curve.
- For curves on low-volume roads with higher speeds (design or estimated operating speed greater than 45 mph [70 km/h]), reconstruction without changing the existing curve geometry and cross section is acceptable if the nominal design speed of the curve is within 10 mph or 20 km/h of the design or operating speed, and if there is no clear evidence of a site-specific crash pattern associated with the curve.

Evidence of a site-specific crash pattern may be demonstrated by a history of curve-related crashes (considering at least 5 years, and preferably 10 years, of crash data); physical evidence of curve problems such as skid marks, scarred trees or utility poles, substantial edge rutting or encroachments, etc.; a history of complaints from residents or local police; or measured or known speeds substantially higher (e.g., 20 mph or 30 km/h higher) than the intended design speed. Even with such evidence, curve improvements should focus on low-cost measures designed to control speeds, enhance curve tracking, or mitigate roadside encroachment severity. Except in rare circumstances, there are more cost effective solutions to identified curve problems on low-volume roads than curve flattening and reconstruction.

Acceptable substitutes for curve reconstruction include measures to reduce speed in the curve (signing, rumble strips, pavement markings), measures to improve the roadside within the curve (clearing slopes, widening shoulder through curve), and measures to increase pavement friction within the curve. Reconstruction employing any or all of these measures should be accompanied by appropriate before-and-after studies to monitor their effectiveness. Procedures for before-and-after evaluation studies are presented in Chapter 9 of the AASHTO *Highway Safety Manual* (**2**).

4.5 STOPPING SIGHT DISTANCE

Sight distance is the length of roadway ahead visible to the driver. The available sight distance on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to avoid colliding with a stationary object in its path. For new construction projects on higher volume roads with design volumes greater than 2,000 vehicles per day, sight distance at every point on the road should be at least that needed for a poorly performing driver or a poorly equipped vehicle to stop within the available sight distance. The object normally considered in stopping sight distance design is a stopped vehicle in the roadway. On local roads with low design volumes (400 vehicles per day or less), on which stopped vehicles would rarely be expected, provision of sufficient sight distance for a driver to maneuver around a small object on the road, rather than come to a stop, may be appropriate.

Stopping sight distance is generally determined as the sum of two distances: (1) the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied; and (2) the distance needed to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively. Similarly, sight distance to maneuver around an object incorporates a maneuver reaction time and a maneuver time. The current stopping sight distance criteria in the AASHTO Green Book (5) are based on the following model:

U.S. Customary	Metric
$SSD = 1.47Vt + 1.075\frac{V^2}{a}$	$SSD = 0.278Vt + 0.039\frac{V^2}{a}$ (4-3)
where: SSD = sight distance, ft; t = brake reaction time, s; V = design speed, mph; and a = deceleration rate, ft/s².	where: SSD = sight distance, m; t = brake reaction time, s; V = design speed, km/h; and a = deceleration rate, m/s².

The brake reaction time (t) of 2.5 s used in Equation 4-3 represents approximately the 95th percentile of the observed distribution of brake-reaction times. The deceleration rate, a, of 11.2 ft/sec^2 [3.4 m/s^2] used in Equation 4-3 represents approximately the 10th percentile of driver deceleration rate. These values of brake reaction time and driver deceleration rate were developed in research for higher volume roads in NCHRP Report 400 (7).

As discussed later in this section, sight distance plays a key role in setting the minimum lengths of crest vertical curves. The AASHTO Green Book (5) uses values for height of eye (h_1) and height of object (h_2) equal to 3.5 ft and 2.0 ft [1,080 mm and 600 mm], respectively.

Sight distance criteria applicable to new construction projects and to existing low-volume roads are presented below. The design criteria for stopping sight distance on low-volume roads vary with traffic volume levels and the proximity of intersections, narrow bridges, railroad-highway grade crossings, sharp curves, and steep grades, but the design criteria do not vary between rural and urban areas or between functional subclasses of low-volume roads.

4.5.1 New Construction

Design of newly constructed low-volume roads with design volumes of 400 vehicles per day or less may be based on sight distances lower than those presented in the AASHTO Green Book (**5**). NCHRP Report 400 (**7**) found that collisions at crest vertical curves with limited sight distance are extremely rare events, even on higher volume roadways, and that the object struck in such collisions was predominately another motor vehicle. Furthermore, there was no indication that lengthening of the sight distance at crest vertical curves has any demonstrable effect on reducing the number of collisions. The risk assessment by Neuman (**11**) for roads with average daily traffic volumes of 400 vehicles per day or less concluded that collisions with vehicles stopped in the roadway were far less likely on such roads than even the limited likelihood of collisions with stopped vehicles on higher volume roads and that sight distance values lower than those presented in the AASHTO Green Book (**5**) for higher volume roads can be applied to such roads with minimal effect on crash frequency and severity. Based on the formal risk assessment by Neuman, two sets of alternative sight distance criteria for roads with design volumes of 400 vehicles per day or less are recommended. The maneuver sight distance model developed in NCHRP Report 400 (**7**) is recommended for application to:

- roads with traffic volumes of 100 vehicles per day or less; or
- roads with traffic volumes of 101 to 250 vehicles per day located at lower risk locations, such as locations not in close proximity to intersections, narrow bridges, railroad–highway grade crossings, sharp curves, or steep downgrades.

The sight distance model presented in Equation 4-3 using alternative parameter values for brake-reaction time and driver deceleration is recommended for the following types of low-volume roads:

- roads with design volumes of 101 to 250 vehicles per day located at higher risk locations, such as locations near intersections, narrow bridges, or railroad–highway grade crossings, or in advance of sharp curves and steep downgrades; or
- roads with design volumes of 251 to 400 vehicles per day

The alternative parameter values recommended for use when Equation 4-3 is applied to new construction of roads with design volumes of 400 vehicles per day or less are:

- a brake-reaction time of 2 s, based on the 90th rather than the 95th percentile of observed driver behavior; or
- a driver deceleration of 13.4 ft/s^2 [4.1 m/s^2], based on the 50th percentile rather than the 10th percentile of the observed distribution

Table 4-7 presents recommended design stopping sight distance criteria for new construction on roads with design volumes of 400 vehicles per day or less based on the models discussed above. These criteria may be used in design of both horizontal and crest vertical curves for new construction.

Table 4-7. Design Stopping Sight Distance Guidelines for New Construction of Low-Volume Roads with Design Volumes of 2,000 Vehicles per Day or Less

U.S. Customary					
	Minimum Sight Distance (ft) for Specified Design Traffic Volumes and Location Types				
	0–100 veh/day	101–250 veh/day		251–400 veh/day	401–2,000 veh/day
Design Speed (mph)	All Locations	"Lower Risk" Locations[1]	"Higher Risk" Locations[2]	All Locations	All Locations
15	65	65	65	65	80
20	90	90	95	95	115
25	115	115	125	125	155
30	135	135	165	165	200
35	170	170	205	205	250
40	215	215	250	250	305
45	260	260	300	300	360
50	310	310	350	350	425
55	365	365	405	405	495
60	435	435	470	470	570

Metric					
	Minimum Sight Distance (m) for Specified Design Traffic Volumes and Location Types				
	0–100 veh/day	101–250 veh/day		251–400 veh/day	401–2,000 veh/day
Design Speed (km/h)	All Locations	"Lower Risk" Locations[1]	"Higher Risk" Locations[2]	All Locations	All Locations
20	15	15	15	15	20
30	25	25	30	30	35
40	35	35	40	40	50
50	45	45	55	55	65
60	60	60	70	70	85
70	75	75	90	90	105
80	95	95	110	110	130
90	120	120	130	130	160
100	140	140	155	155	185

[1] Not in close proximity to intersections, narrow bridges, railroad–highway grade crossings, sharp curves, or steep downgrades.
[2] Near intersections, narrow bridges, or railroad–highway grade crossings, or in advance of sharp curves or steep downgrades.

For new construction of roads with design volumes greater than 400 vehicles per day, the stopping sight distance criteria presented in Chapter 3 of the AASHTO Green Book (5) should be applied.

4.5.1.1 Sight Distance on Horizontal Curves

Sight distance across the inside of horizontal curves is an element of the design of horizontal alignment. Where there are sight obstructions (such as walls, cut slopes, vegetation, buildings, or longitudinal barriers) on the inside of a horizontal curve, a design to provide adequate sight distance may need an adjustment in the normal highway cross section or a change in alignment if the obstruction cannot be removed. Because of the many variables in alignment and cross sections and in the number, type, and location of possible obstructions, a specific study is usually needed for each condition. With the sight distance specified in Table 4-7 for the appropriate design speed as a control, the designer should check the actual condition and make any needed adjustments in the manner most fitting to provide adequate sight distances.

For general use in the design of a horizontal curve, the sight line is a chord of the horizontal curve, and the applicable stopping sight distance is measured along the centerline of the inside lane around the curve. The minimum width that should be clear of sight obstructions is the middle ordinate of the curve, referred to in geometric design as the horizontal sightline offset, *HSO*, as shown in Figure 4-1.

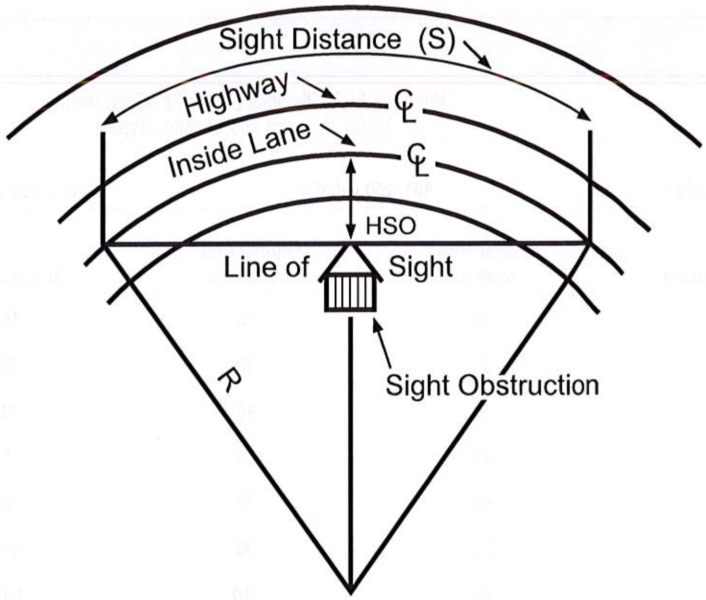


Figure 4-1. Horizontal Curve Showing Stopping Sight Distance Along the Curve and the Horizontal Sightline Offset that Defines the Maximum Unobstructed Width

The horizontal sightline offset can be computed, for any curve whose length exceeds the design sight distance, as shown in Equation 4-4:

U.S. Customary	Metric	
$$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right]$$	$$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right]$$	(4-4)
where: S = Sight distance, ft; R = Radius of curve, ft; and HSO = Horizontal sightline offset, ft.	where: S = Sight distance, m; R = Radius of curve, m; and HSO = Horizontal sightline offset, m.	

Table 4-8 presents the horizontal sightline offsets that define the width that should be clear of sight obstructions for a horizontal curve as a function of curve radius and design speed.

Table 4-8. Design Guidelines for Sight Distance on Horizontal Curves for New Construction of Low-Volume Roads

U.S. Customary

All Locations for 0–100 veh/day and "Lower Risk" Locations for 101–250 veh/day[1]

Design Speed (mph)	Stopping Sight Distance (ft)	Width on Inside of Curve Clear of Sight Obstructions[3] (ft) — Radius of Curvature (ft): 50	100	200	500	1000	2000	5000	10000	20000
15	65	10.2	5.2	2.6	1.1	0.5	0.3	0.1	0.1	0.0
20	90	—	10.0	5.0	2.0	1.0	0.5	0.2	0.1	0.1
25	115	—	—	8.2	3.3	1.7	0.8	0.3	0.2	0.1
30	135	—	—	11.3	4.5	2.3	1.1	0.5	0.2	0.1
35	170	—	—	—	7.2	3.6	1.8	0.7	0.4	0.2
40	215	—	—	—	11.5	5.8	2.9	1.2	0.6	0.3
45	260	—	—	—	16.8	8.4	4.2	1.7	0.8	0.4
50	310	—	—	—	—	12.0	6.0	2.4	1.2	0.6
55	365	—	—	—	—	16.6	8.3	3.3	1.7	0.8
60	435	—	—	—	—	23.6	11.8	4.7	2.4	1.2

"Higher Risk" Locations for 101–250 veh/day and All Locations for 251–400 veh/day[2]

Design Speed (mph)	Stopping Sight Distance (ft)	Width on Inside of Curve Clear of Sight Obstructions[3] (ft) — Radius of Curvature (ft): 50	100	200	500	1000	2000	5000	10000	20000
15	65	10.2	5.2	2.6	1.1	0.5	0.3	0.1	0.1	0.0
20	95	—	11.1	5.6	2.3	1.1	0.6	0.2	0.1	0.1
25	125	—	—	9.7	3.9	2.0	1.0	0.4	0.2	0.1
30	165	—	—	16.8	6.8	3.4	1.7	0.7	0.3	0.2
35	205	—	—	—	10.5	5.2	2.6	1.1	0.5	0.3
40	250	—	—	—	15.5	7.8	3.9	1.6	0.8	0.4
45	300	—	—	—	22.3	11.2	5.6	2.3	1.1	0.6
50	350	—	—	—	—	15.3	7.7	3.1	1.5	0.8
55	405	—	—	—	—	20.4	10.2	4.1	2.1	1.0
60	470	—	—	—	—	27.5	13.8	5.5	2.8	1.4

Metric

All Locations for 0–100 veh/day and "Lower Risk" Locations for 101–250 veh/day[1]

Design Speed (km/h)	Stopping Sight Distance (m)	Width on Inside of Curve Clear of Sight Obstructions[3] (m) — Radius of Curvature (m): 10	50	100	200	500	1000	2000	4000	6000
20	15	2.7	0.6	0.3	0.1	0.1	0.0	0.0	0.0	0.0
30	25	—	1.6	0.8	0.4	0.2	0.1	0.0	0.0	0.0
40	35	—	3.0	1.5	0.8	0.3	0.2	0.1	0.0	0.0
50	45	—	—	2.5	1.3	0.5	0.3	0.1	0.1	0.0
60	60	—	—	—	2.2	0.9	0.5	0.2	0.1	0.1
70	75	—	—	—	3.5	1.4	0.7	0.4	0.2	0.1
80	95	—	—	—	5.6	2.3	1.1	0.6	0.3	0.2
90	120	—	—	—	—	3.6	1.8	0.9	0.5	0.3
100	140	—	—	—	—	4.9	2.4	1.2	0.6	0.4

"Higher Risk" Locations for 101–250 veh/day and All Locations for 251–400 veh/day[2]

Design Speed (km/h)	Stopping Sight Distance (m)	Width on Inside of Curve Clear of Sight Obstructions[3] (m) — Radius of Curvature (m): 10	50	100	200	500	1000	2000	4000	6000
20	15	2.7	0.6	0.3	0.1	0.1	0.0	0.0	0.0	0.0
30	30	—	2.2	1.1	0.6	0.2	0.1	0.1	0.0	0.0
40	40	—	3.9	2.0	1.0	0.4	0.2	0.1	0.1	0.0
50	55	—	—	3.8	1.9	0.8	0.4	0.2	0.1	0.1
60	70	—	—	—	3.1	1.2	0.6	0.3	0.2	0.1
70	90	—	—	—	5.0	2.0	1.0	0.5	0.3	0.2
80	110	—	—	—	7.5	3.0	1.5	0.8	0.4	0.3
90	130	—	—	—	—	4.2	2.1	1.1	0.5	0.4
100	155	—	—	—	—	6.0	3.0	1.5	0.8	0.5

Continued on next page.

Table 4-8. Design Guidelines for Sight Distance on Horizontal Curves for New Construction of Low-Volume Roads (Continued)

U.S. Customary

All Locations for 401–2,000 veh/day

Design Speed (mph)	Stopping Sight Distance (ft)	Width on Inside of Curve Clear of Sight Obstructions[3] (ft) — Radius of Curvature (ft): 50	100	200	500	1,000	2,000	5,000	10,000	20,000
15	80	15.2	7.9	4.0	1.6	0.8	0.4	0.2	0.1	0.0
20	115	—	16.1	8.2	3.3	1.7	0.8	0.3	0.2	0.1
25	155	—	—	14.8	6.0	3.0	1.5	0.6	0.3	0.2
30	200	—	—	24.5	10.0	5.0	2.5	1.0	0.5	0.3
35	250	—	—	—	15.5	7.8	3.9	1.6	0.8	0.4
40	305	—	—	—	23.1	11.6	5.8	2.3	1.2	0.6
45	360	—	—	—	32.1	16.2	8.1	3.2	1.6	0.8
50	425	—	—	—	—	22.3	11.3	4.5	2.3	1.1
55	495	—	—	—	—	30.5	15.3	6.1	3.1	1.5
60	570	—	—	—	—	40.3	20.3	8.1	4.1	2.0

Metric

All Locations for 401–2,000 veh/day

Design Speed (km/h)	Stopping Sight Distance (m)	Width on Inside of Curve Clear of Sight Obstructions[3] (m) — Radius of Curvature (m): 10	50	100	200	500	1,000	2,000	4,000	6,000
20	20	4.6	1.0	0.5	0.2	0.1	0.1	0.0	0.0	0.0
30	35	—	3.0	1.5	0.8	0.3	0.2	0.1	0.0	0.0
40	50	—	6.1	3.1	1.6	0.6	0.3	0.2	0.1	0.1
50	65	—	—	5.2	2.6	1.1	0.5	0.3	0.1	0.1
60	85	—	—	—	4.5	1.8	0.9	0.5	0.2	0.2
70	105	—	—	—	6.9	2.8	1.4	0.7	0.3	0.2
80	130	—	—	—	10.5	4.2	2.1	1.1	0.5	0.4
90	160	—	—	—	—	6.4	3.2	1.6	0.8	0.5
100	185	—	—	—	—	8.5	4.3	2.1	1.1	0.7

[1] "Lower risk" locations are locations not in close proximity to intersections, narrow bridges, railroad–highway grade crossings, sharp curves, or steep downgrades.

[2] "Higher risk" locations are locations near intersections, narrow bridges, or railroad–highway grade crossings, or in advance of sharp curves or steep downgrades.

[3] Width on inside of curve clear of sight obstructions is measured from the centerline of the inside lane.

4.5.1.2 Sight Distance on Vertical Curves

Vertical curves are provided to effect a smooth and gradual change between tangent grades and may be any one of the crest or sag types depicted in Figure 4-2. Vertical curves should be simple in application and should result in a design that is comfortable in operation, pleasing in appearance, and adequate for drainage, with limited likelihood of crashes. For simplicity, the parabolic curve with an equivalent vertical axis centered on the vertical point of intersection is usually used in roadway profile design. The vertical offsets from the tangent vary as the square of the horizontal distance from the beginning of the curve.

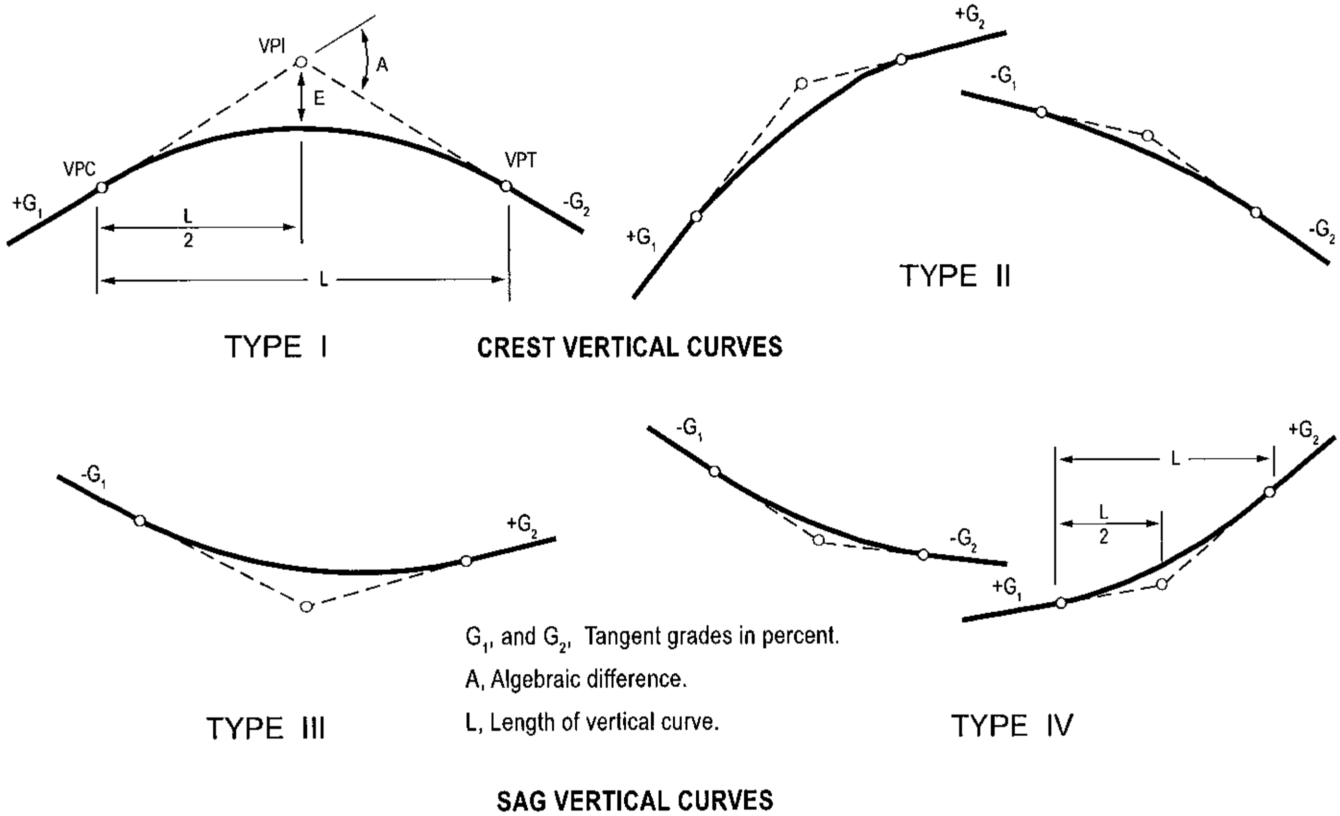


Figure 4-2. Types of Vertical Curves

4.5.1.3 Crest Vertical Curves

The major control for operation on crest vertical curves is the provision of sight distance appropriate for the roadway design speed. In new construction of low-volume roads, crest vertical curves should, where practical, be designed to have at least the length that provides the stopping sight distance values presented in Table 4-7. These lengths can be determined as shown in Equations 4-5 through 4-8:

U.S. Customary	Metric	
When S is less than L, $$L = \frac{AS^2}{100\left(\sqrt{2h_1}+\sqrt{2h_2}\right)^2}$$	When S is less than L, $$L = \frac{AS^2}{100\left(\sqrt{2h_1}+\sqrt{2h_2}\right)^2}$$	(4-5)
When S is greater than L, $$L = 2S - \frac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A}$$	When S is greater than L, $$L = 2S - \frac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A}$$	(4-6)
where: L = length of vertical curve, ft; S = sight distance, ft; A = algebraic difference in grades, percent; h_1 = height of eye above roadway surface, ft; and h_2 = height of object above roadway surface, ft.	where: L = length of vertical curve, m; S = sight distance, m; A = algebraic difference in grades, percent; h_1 = height of eye above roadway surface, m; and h_2 = height of object above roadway surface, m.	

When the height of eye (h_1) and height of object (h_2) are 3.5 ft and 2.0 ft [1,080 mm and 600 mm], respectively, as used for stopping sight distance, Equations 4-5 and 4-6 become:

U.S. Customary	Metric	
When S is less than L, $$L = \frac{AS^2}{2158}$$	When S is less than L, $$L = \frac{AS^2}{658}$$	(4-7)
When S is greater than L, $$L = 2S - \frac{2158}{A}$$	When S is greater than L, $$L = 2S - \frac{658}{A}$$	(4-8)

Table 4-9 presents the rate of vertical curvature, K, that will provide stopping sight distance for crest vertical curves on low-volume roads. The appropriate length for a vertical curve can generally be determined by multiplying the K-value in Table 4-9 by the algebraic difference in grade between the adjoining tangents.

Table 4-9. Guidelines for Minimum Rate of Vertical Curvature to Provide Design Stopping Sight Distance on Crest Vertical Curves for New Construction of Low-Volume Roads

U.S. Customary

	All Locations for 0–100 veh/day and "Lower Risk" Locations for 101–250 veh/day[1]			"Higher Risk" Locations for 101–250 veh/day and All Locations for 251–400 veh/day[2]			All Locations for 401–2,000 veh/day		
		Rate of Vertical Curvature, K[3]			Rate of Vertical Curvature, K[3]			Rate of Vertical Curvature, K[3]	
Design Speed (mph)	Stopping Sight Distance (ft)	Calculated	Design	Stopping Sight Distance (ft)	Calculated	Design	Stopping Sight Distance (ft)	Calculated	Design
15	65	2.0	2	65	2.0	2	80	3.0	3
20	90	3.8	4	95	4.2	5	115	6.1	7
25	115	6.1	7	125	7.2	8	200	11.1	12
30	135	8.4	9	165	12.6	13	250	18.5	19
35	170	13.4	14	205	19.5	20	305	29.0	29
40	215	21.4	22	250	29.0	29	360	43.1	44
45	260	31.3	32	300	41.7	42	425	60.1	61
50	310	44.5	45	350	56.8	57	495	83.7	84
55	365	61.7	62	405	76.0	76	570	113.5	114
60	435	87.7	88	470	102.4	103	—	150.6	151

Metric

	All Locations for 0–100 veh/day and "Lower Risk" Locations for 101–250 veh/day[1]			"Higher Risk" Locations for 101–250 veh/day and All Locations for 251–400 veh/day[2]			All Locations for 401–2,000 veh/day		
		Rate of Vertical Curvature, K[3]			Rate of Vertical Curvature, K[3]			Rate of Vertical Curvature, K[3]	
Design Speed (km/h)	Stopping Sight Distance (m)	Calculated	Design	Stopping Sight Distance (m)	Calculated	Design	Stopping Sight Distance (m)	Calculated	Design
20	15	0.3	0.5	15	0.3	0.5	20	0.6	1
30	25	0.9	1	30	1.4	2	35	1.9	2
40	35	1.9	2	40	2.4	4	50	3.8	4
50	45	3.1	4	55	4.6	5	65	6.4	7
60	60	5.5	6	70	7.4	8	85	11.0	11
70	75	8.5	9	90	12.3	13	105	16.8	17
80	95	13.7	14	110	18.4	19	130	25.7	26
90	120	21.9	22	130	25.7	26	160	38.9	39
100	140	29.8	30	155	36.5	37	185	52.0	52

[1] "Lower risk" locations are locations not in close proximity to intersections, narrow bridges, railroad–highway grade crossings, sharp curves, or steep downgrades.

[2] "Higher risk" locations are locations near intersections, narrow bridges, or railroad–highway grade crossings, or in advance of sharp curves or steep downgrades.

[3] The rate of vertical curvature, K, is the length of curve (L) per percent algebraic difference in intersecting grades (A); i.e., $K = L/A$.

4.5.1.4 Sag Vertical Curves

There are no special guidelines for design of sag vertical curves on low-volume roads. Sag vertical curves should generally be designed in accordance with Chapters 5 and 6 of the AASHTO Green Book (**5**).

4.5.2 Existing Roads

Given the geometry of stopping sight distance on horizontal and crest vertical curves, the costs for even marginal or incremental improvements make reconstruction of low-volume roads to increase stopping sight distance not cost-effective except in unusual cases. NCHRP Report 400 (**7**) found that, even on higher volume roadways, crashes associated with limited sight distance are extremely rare events. Furthermore, there was no indication that lengthening the sight distance of a crest vertical curve has any demonstrable effect on reducing the number of collisions. Collisions related to limited sight distance are even less likely on low-volume roads than on the higher volume roads studied in NCHRP Report 400 (**7**).

Because sight distance improvements are unlikely to be cost-effective under most circumstances, the existing sight distance on a low-volume road may be allowed to remain in place unless there is evidence of a site-specific crash pattern attributable to inadequate sight distance. If a site-specific crash pattern is identified, and if the designer finds after investigation that the crash pattern is attributable to limited sight distance, then the sight distance of the specific horizontal or vertical curve(s) at which the problem is present should be upgraded to at least the sight distance levels shown in Table 4-7 as part of any reconstruction project undertaken. Sight distance could be increased to the full criteria presented in the AASHTO Green Book (**5**) where the judgment of the designer indicates that this is appropriate. This approach is intended to provide maximum flexibility to the designer in assessing site-specific conditions and exercising informed judgment to decide whether a correctable problem is present or not. Guidance concerning identification of site-specific crash patterns is found in Section 3.5 of these guidelines.

4.6 INTERSECTION SIGHT DISTANCE

4.6.1 General Considerations

Each intersection has the potential for several different types of vehicle–vehicle conflicts. The possibility of these conflicts actually occurring can be greatly reduced through the provision of proper sight distances and appropriate traffic controls. The avoidance of crashes and the efficiency of traffic operations still depend on the judgment, capabilities, and response of each individual driver.

The driver of a vehicle approaching an at-grade intersection should have an unobstructed view of the entire intersection, including any intersection traffic-control devices, and sufficient lengths of the intersecting road to permit the driver to anticipate and avoid potential collisions. The sight distance that should be used for design under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and to the resultant distances traversed during perception–reaction time and braking.

Guidelines for intersection sight distance at intersections between low-volume roads with design volumes of 400 vehicles per day or less are presented here. However, if one or more of the intersection legs has a design traffic volume that exceeds 400 vehicles per day, intersection sight distance for newly constructed intersections should be designed in accordance with Chapter 9 of the AASHTO Green Book (**5**).

Stopping sight distance is generally provided continuously along each road or street so that drivers have a view of the roadway ahead that is sufficient to allow drivers to stop or take evasive action, if necessary, under prescribed conditions. The provision of stopping sight distance at all locations along each road or street, including intersection approaches, is fundamental to intersection operations.

Vehicles are assigned the right-of-way at intersections by traffic-control devices or, where no traffic-control devices are present, by the rules of the road. A basic rule of the road is that, at an intersection at which no traffic-control devices are present, the vehicle on the left must yield the right-of-way to the vehicle on the right if they arrive at approximately the same time. Sight distance is provided at intersections to allow the drivers of vehicles without the right-of-way to perceive the presence of potentially conflicting vehicles in sufficient time for the vehicle without the right-of-way to stop, if necessary, before reaching the intersection. The methods for determining the sight distances needed by drivers approaching intersections are based on the same principles as stopping sight distance, but incorporate modified assumptions based on observed driver behavior at intersections.

Sight distance is also provided at intersections to allow the drivers of vehicles stopped on intersection approaches a sufficient view of the intersecting highway to decide when to turn onto the intersecting highway or to cross it from a stop- or yield-controlled approach to an intersection that has both controlled and uncontrolled approaches. If the available sight distance for an entering or crossing vehicle is at least equal to the appropriate stopping sight distance for the uncontrolled approach, then drivers should have sufficient sight distance to anticipate and avoid collisions. However, in some cases, this may require a vehicle on an uncontrolled approach to stop or slow to accommodate a turning maneuver by a vehicle from a controlled approach. Intersections between two low-volume roads with design volumes of 400 vehicles per day or less can be operated effectively with approach sight distances based on stopping sight distances. To achieve better traffic operations, so that vehicles on uncontrolled approaches do not need to stop or slow substantially to accommodate entering or crossing vehicles, intersection sight distances that exceed stopping sight distance are desirable along the uncontrolled approaches. Thus, intersection sight distances that exceed stopping sight distance are intended to enhance traffic operations, but are not minimum design criteria that are essential to limiting crash risk.

4.6.2 Clear Sight Triangles

Specified areas along intersection approach legs and across their included corners should be clear of obstructions that might block a driver's view of potentially conflicting vehicles. These specified areas are known as clear sight triangles. Two types of clear sight triangles considered in intersection design, approach sight triangles and departure sight triangles, are explained below. The dimensions of the clear sight triangles depend on the design speeds of the intersecting roadways and the type of traffic control used at the intersection. These dimensions are based on field studies in NCHRP Report 383 (**9**) that have observed driver behavior and have documented the space–time profiles and speed choices of drivers on intersection approaches.

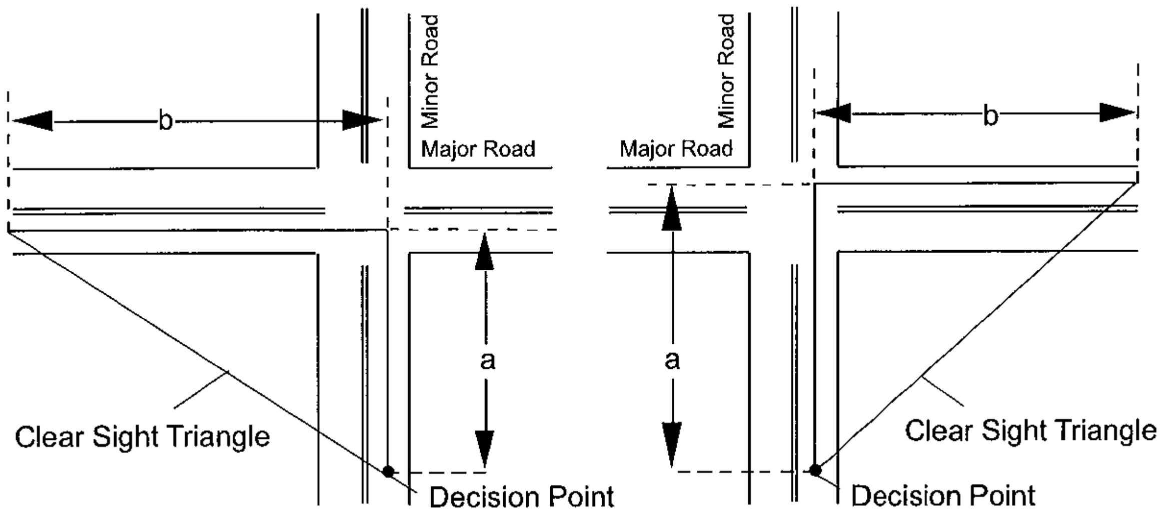
4.6.2.1 Approach Sight Triangles

Each quadrant of an uncontrolled or yield-controlled intersection should contain a clear sight triangle free of obstructions that might block an approaching driver's view of potentially conflicting vehicles on the intersecting approaches. The area clear of sight obstructions should include sufficient lengths of both intersecting roadways, as well as their included corner, so that the drivers without the right-of-way can see any potentially conflicting vehicle in sufficient time to slow or stop before reaching the intersection. Figure 4-3A shows typical clear sight triangles to the left and to the right for a vehicle approaching an intersection.

The vertex of the sight triangle on the uncontrolled or yield-controlled approach represents a decision point for the approaching driver. This decision point is the location at which the driver should begin to brake to a stop if another vehicle is present on an intersecting approach. The distance from the decision point to the center of the major-road lane into which a driver will turn is shown in Figure 4-3A as distance *a*. The length of the leg of the clear sight triangle along the major road is shown in Figure 4-3A as distance *b*.

The geometry of a clear sight triangle is such that when the driver of a vehicle without the right-of-way sees a potentially conflicting vehicle on an intersecting approach that has the right of-way, then the driver of that potentially conflicting vehicle can also see the first vehicle. Thus, the provision of a clear sight triangle for vehicles without the right-of-way also permits the drivers of vehicles with the right-of-way to be prepared to slow, stop, or avoid other vehicles, should it become necessary.

Approach sight triangles like those shown in Figure 4-3A are not needed for intersection approaches controlled by stop signs because all approaching vehicles are required to stop at the intersection, regardless of the presence or absence of vehicles on the intersecting approaches.



Clear Sight Triangle for Viewing
Traffic Approaching from the Left

Clear Sight Triangle for Viewing
Traffic Approaching from the Right

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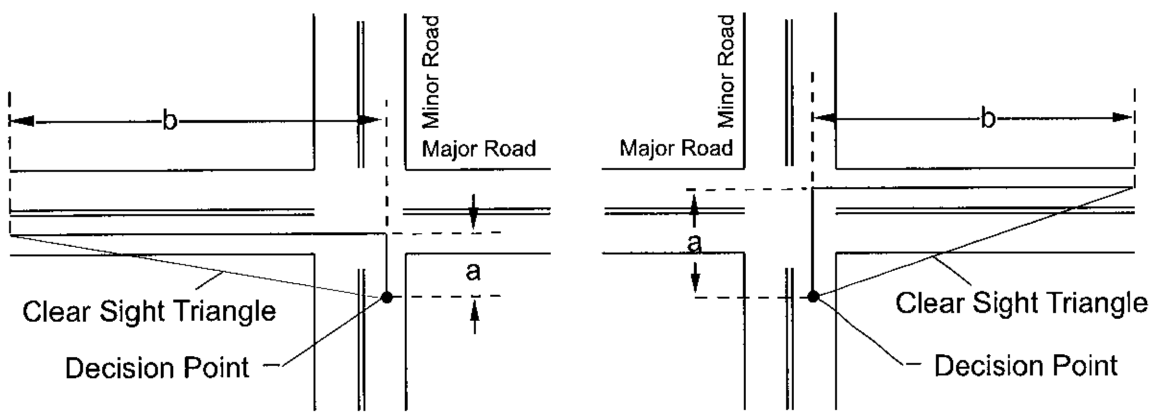
A—Approach Sight Triangles



Clear Sight Triangle for Viewing
Traffic Approaching from the Left

Clear Sight Triangle for Viewing
Traffic Approaching from the Right

B—Departure Sight Triangles

Figure 4-3. Clear Sight Triangles for Intersection Approaches

4.6.2.2 Departure Sight Triangles

A second type of clear sight triangle provides sight distance sufficient for a driver stopped on a stop- or yield-controlled approach to depart from the intersection by entering or crossing the intersecting road. Figure 4-3B shows typical departure sight triangles to the left and to the right. The distance from the stopped vehicle position or decision point to the center of the major-road lane into which a driver will turn is shown in Figure 4-3B as distance *a*. The length of the leg of the clear sight triangle along the major road is shown in Figure 4-3B as distance *b*. Departure sight triangles should be provided in each quadrant of each intersection approach controlled by stop or yield signs from which stopped vehicles may enter or cross a road on which traffic is not required to stop.

4.6.2.3 Identification of Sight Obstructions within Clear Sight Triangles

The profiles of the intersecting roadways should be designed to provide the recommended sight distances for drivers on the intersection approaches. Within a clear sight triangle, any object at a height above the elevation of the adjacent roadways that would obstruct the driver's view should be removed or lowered, if practical. Such objects may include: buildings, parked vehicles, highway structures, roadside hardware, hedges, trees, bushes, unmowed grass, tall crops, and the terrain itself.

The determination of whether an object constitutes a sight obstruction should consider the horizontal and vertical alignment of both intersecting roadways, as well as the height and position of the object. In making this determination, it should be assumed that the driver's eye is 3.5 ft [1,080 mm] above the roadway surface and that the object to be seen is also 3.5 ft [1,080 mm] above the surface of the intersecting road. This object height is based on a vehicle height of 4.4 ft [1,330 mm], which represents the 15th percentile of vehicle heights in the current passenger car population less an allowance of 0.9 ft [250 mm], which represents a near-maximum value for the portion of the vehicle height that needs to be visible for another driver to recognize a vehicle as such. The use of an object height equal to the driver eye height makes intersection sight distances reciprocal (i.e., if one driver can see another vehicle, then the driver of that vehicle can also see the first vehicle).

4.6.3 New Construction

Sight distance design for newly constructed intersections at which all intersection legs are low-volume roads with design volumes of 400 vehicles per day or less should be based on the criteria presented below. If one or more of the intersection legs has a design volume that exceeds 400 vehicles per day, the sight distance criteria in Chapter 9 of the AASHTO Green Book (**5**) should be applied.

The sight distance design criteria for intersections between low-volume roads vary with the type of traffic control used at an intersection because different types of control impose different legal constraints on drivers and, therefore, result in different driver behavior. Sight distance policies for intersections with the following types of traffic control are presented below:

- Intersections with no control (Case A),
- Intersections with stop control on the minor road (Case B), and
- Intersections with yield control on the minor road (Case C).

Other intersection sight distance cases are presented in the AASHTO Green Book (**5**).

4.6.3.1 Intersections with No Control (Case A)

For intersections not controlled by yield signs, stop signs, or traffic signals, the driver of a vehicle approaching the intersection should be able to see potentially conflicting vehicles on intersecting approaches in sufficient time for the approaching driver to stop before reaching the intersection. The location of the vertex of the sight triangles on each approach is determined from a model that is analogous to the stopping sight distance model, with slightly different assumptions. Drivers of approaching vehicles may need up to 2.5 s to perceive vehicles on intersecting approaches and to initiate braking.

While some perceptual tasks at intersections may need substantially less time, the detection and recognition of a vehicle that is a substantial distance away on an intersecting approach, and is near the limits of the driver's peripheral vision, may need up to 2.5 s. The distance to brake to a stop can be determined from the same braking coefficient used for stopping sight distance design.

Field observations in NCHRP Report 383 (**9**) indicate that vehicles approaching uncontrolled intersections typically slow down from their running speed between intersections to approximately 50 percent of their running speed. This occurs even when no potentially conflicting vehicles are present. This initial slowing typically occurs at deceleration rates up to 5 ft/s^2 [1.5 m/s^2], deceleration at this gradual rate has been observed to begin even before a potentially conflicting vehicle comes into view. Braking at greater deceleration rates, which can approach those assumed in stopping sight distance, begins up to 2.5 s after a vehicle on the intersecting approach comes into view. Thus, approaching vehicles may be traveling at less than their running speed upstream of the intersection during all or part of the perception–reaction time and can, therefore, where necessary, brake to a stop from a speed less than the running speed upstream of the intersection.

Table 4-10 shows the distance traveled by an approaching vehicle during perception–reaction and braking time as a function of the design speed of the roadway on which the intersection approach is located. These distances should be used as the legs of the sight triangles shown in Figure 4-3A. Referring to Figure 4-3A, roadway A with a 50 mph [80 km/h] design speed and roadway B with a 30 mph [50 km/h] design speed need a clear sight triangle with legs extending at least 225 ft [80 m] and 120 ft [40 m] along roadways A and B, respectively.

This clear sight triangle will permit the vehicles on either road to stop, if necessary, before reaching the intersection. If the design speed of any approach is not known, it can be estimated by using the 85th percentile of the running speeds upstream of the intersection on that intersection leg.

The distances shown in Table 4-10 are generally less than the corresponding values of stopping sight distance for the same design speed. Where a clear sight triangle whose legs correspond to the stopping sight distances of their respective approaches can be provided, this will likely reduce crash frequency and severity even further. However, since field observations show that motorists slow down to some extent on approaches to uncontrolled intersections, the provision of a clear sight triangle with legs equal to the full stopping sight distance is not essential.

Where the grade along an intersection approach exceeds 3 percent, the leg of the clear sight triangle along that approach should be adjusted by multiplying the appropriate sight distance from Table 4-10 by the appropriate adjustment factor from Table 4-11.

If the sight distances given in Table 4-10, as adjusted for grades, cannot be provided, consideration should be given to installing advisory speed signing to reduce speeds or installing stop signs on one or more approaches.

No departure sight triangle like that shown in Figure 4-3B is needed at an uncontrolled intersection because of the very low traffic volumes present on the intersection approaches, typically less than 400 vehicles per day.

If a motorist finds it necessary to stop at an uncontrolled intersection because of the presence of a conflicting vehicle on an intersecting approach, it is unlikely that another potentially conflicting vehicle will be encountered as the first vehicle departs the intersection.

Table 4-10. Recommended Sight Distance Guidelines for New Construction of Intersections with No Traffic Control (Case A) (5, 17)

U.S. Customary		Metric	
Design Speed (mph)	Sight Distance (ft)	Design Speed (km/h)	Sight Distance (m)
15	60	20	20
20	80	30	25
25	95	40	30
30	120	50	40
35	140	60	50
40	170	70	65
45	210	80	80
50	255	90	95
55	300	100	120
60	350		

Note: For approach grades greater than 3 percent, multiply the sight distance value by the appropriate adjustment factor from Table 4-11.

Table 4-11. Adjustment Factors for Sight Distance Based on Approach Grade (5, 9)

U.S. Customary										
Approach Grade (%)	Design Speed (mph)									
	15	20	25	30	35	40	45	50	55	60
−6	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
−5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
−4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
−3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Metric									
Approach Grade (%)	Design Speed (km/h)								
	20	30	40	50	60	70	80	90	100
−6	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
−5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
−4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1
−3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Note: Based on ratio of stopping sight distance on specified approach grade to stopping sight distance on level terrain.

4.6.3.2 Intersections with Stop Control on the Minor Road (Case B)

No approach sight triangles like those shown in Figure 4-3A are needed on stop-controlled approaches because all vehicles on the approach are required to stop before entering or crossing the intersecting road.

Departure sight triangles to the left and the right like those shown in Figure 4-3B should be provided for each stop- or yield-controlled approach. Whenever practical, a leg of the departure sight triangle along each uncontrolled approach equal to at least the full intersection sight distance for stop-controlled intersections, as presented in Chapter 9 of the AASHTO Green Book (**5**), should be provided. In constrained situations, the length of the leg of the departure triangle along the major road should be at least equal to the stopping sight distance appropriate for the design speed of the major road as determined from Table 4-7. For the design volume range from 100 to 250 vehicles per day, the sight distances in the column of Table 4-7 headed "higher risk" locations should be used, because this column is appropriate for application to intersections. The vertex of the departure sight triangle on the minor road should be 14.4 ft [4.4 m] from the edge of the major-road traveled way (**5, 9**).

4.6.3.3 Intersections with Yield Control on the Minor Road (Case C)

Approach sight triangles to the left and to the right like those shown in Figure 4-3A should be provided for each yield-controlled intersection approach. Whenever practical, legs of the approach sight triangles equal to at least the full intersection sight distances for yield-controlled intersections, as presented in Chapter 9 of the AASHTO Green Book (**5**), should be provided. In constrained situations, the leg of the approach sight triangle along each intersection approach should be at least equal to the stopping sight distance appropriate for the design speed of that approach as determined from Table 4-7. For the design volume range from 100 to 250 vehicles per day, the sight distances in the column of Table 4-7 headed "higher risk" locations should be used because this column is appropriate for application to intersections. The grade adjustment factors in Table 4-11 also apply to this case.

No separate departure sight triangles for yield-controlled intersections need be considered. The approach sight triangles for yield-controlled intersections described above include departure sight triangles equivalent to those described earlier for stop-controlled intersections on low-volume roads.

4.6.4 Existing Roads

For improvement projects at existing intersections between low-volume roads, the existing intersection sight distance may generally remain in place unless there is evidence of a site-specific crash pattern related to intersection sight distance. Where there is evidence of a site-specific crash pattern, the intersection sight distance should be increased to at least the appropriate values shown above for new construction.

4.7 ROADSIDE DESIGN

Two key aspects of roadside design are clear zone width and traffic barrier warrants. AASHTO policy on these aspects of roadside design for higher volume roads is presented in the AASHTO *Roadside Design Guide* (**3**). This section presents guidelines for roadside design on low-volume roads that may be used in lieu of these other AASHTO policies and guidelines. For design issues not addressed in this guide, the designer should consult the applicable sections of these other AASHTO policies and guidelines.

A clear zone is that portion of the roadside that is free of obstructions and sufficiently flat to enable an errant vehicle to encroach without overturning. The clear zone width at any point along the roadway is measured from the edge of the traveled way to the nearest obstruction or the beginning of a non-traversable slope. Thus, shoulders are part of the roadside clear zone.

A traffic barrier is a device used to prevent a vehicle from striking a more severe obstacle or feature located on the roadside. Traffic barriers include roadside barriers, median barriers, bridge railings, and crash cushions.

The roadside design is the one major determinant of crash frequency and severity on low-volume roads, if for no other reason than that multiple-vehicle collisions on the roadway are rare. Both the safety literature and the risk assessment conducted by Neuman (**11**) indicate that run-off-road crashes on roads with design volumes of 400 vehicles per day or less occur so infrequently as to make any minimum clear zone width demonstrably not cost-effective. In many cases, the provision of additional clear zone width increases construction costs and involves additional right-of-way acquisition which potentially has both cost and environmental concerns.

Research has found that roadside clear zones and traffic barriers are not generally cost-effective on roads with low traffic volumes, particularly for roads with design volumes of 400 vehicles per day or less (12, 17, 18). However, there are no established criteria to identify those limited situations where provision of a roadside clear zone or a traffic barrier may be warranted. Therefore, the roadside design guidelines for low-volume roads provide great flexibility to the designer in exercising engineering judgment to decide where it is appropriate to provide improved roadsides.

4.7.1 New Construction

Roadside design guidelines applicable to new construction of low-volume roads are presented below. The guidelines address both clear zone width and traffic barrier warrants and are appropriate for all functional subclasses of low-volume roads.

4.7.1.1. Clear Zone Width

The risk assessment discussed in Section 3.4 of this guide found that it is not generally cost-effective to provide clear zones, also known as clear recovery areas, on low-volume roads. Nevertheless, a clear zone of any width should provide some contribution to crash reduction. Thus, where clear zones can be provided on low-volume roads at little or no additional cost, their incorporation in designs should be considered. Clear zones may also be appropriate on horizontal curves where the minimum radius of curvature is not provided. However, major expenditures to provide clear zones will generally have only limited crash reduction benefits and are unlikely to be cost-effective. The design guidelines for roadside clear zone width on low-volume rural roads with design volumes of 400 vehicles per day or less are as follows:

1. At locations where a clear recovery area of 6 ft [2 m] or more in width can be provided at low cost and with minimum social or environmental impacts, provision of such a clear recovery area should be considered.
2. Where constraints of cost, terrain, right-of-way, or potential social or environmental impacts make the provision of a 6-ft [2-m] clear recovery area impractical, clear recovery areas less than 6 ft [2 m] in width may be used, including designs with 0 ft [0 m] clear recovery areas.
3. In all cases, designers should be encouraged to tailor the roadside design to site-specific conditions, considering cost-effectiveness and crash risk tradeoffs. For example, the use of adjustable clear zone widths may be appropriate in some cases, such as providing wider clear zone dimensions at sharp horizontal curves where there is a history of run-off-road crashes or where scarring of trees or utility poles may indicate possible vehicle encroachments. Lesser values of clear zone width may be appropriate on tangent sections of the same roadway.
4. Other factors for consideration in analyzing the need for providing clear zones include the crash history, the expectation for future traffic volume growth on the facility, and the presence of vehicles wider than 8.5 ft [2.6 m] and vehicles with wide loads, such as farm equipment.

On low-volume rural roads with design volumes from 401 to 2,000 vehicles per day, clear zones with widths of 7 to 10 ft [2 to 3 m] are desirable.

Provision of clear zones is often not practical for urban low-volume streets. On urban local streets, clear zones are not generally provided. On urban minor collector streets, designs with reduced clear zones or designs incorporating as many roadside safety features as practical may be considered.

4.7.1.2 Traffic Barriers

The use of guardrail or other traffic barriers to protect drivers from roadside obstructions is not generally cost-effective for roads with design volumes of 400 vehicles per day or less. This finding has been confirmed in studies by Stephens (**12**) and Wolford and Sicking (**18**). Guardrail itself is a roadside obstacle, and a significant proportion of vehicle impacts with guardrail produce injuries. The costs to maintain guardrail and the low frequency of collisions with guardrail that is provided generally make it impractical for use on roads with low traffic volumes. For low-volume roads with design volumes above 400 vehicles per day, designers may exercise engineering judgment concerning the placement of guardrail at locations where the potential consequences of departure from the roadway are likely to be extremely severe.

4.7.2 Existing Roads

The roadside design guidelines for existing low-volume roads are the same as those for newly constructed roads. Roadside clear zones and traffic barriers are not generally cost effective and need not generally be provided, except in situations where a site-specific crash pattern is present or the engineering judgment of the designer identifies a need for the provision of a roadside clear zone or a guardrail. Evidence of a site-specific crash pattern that could indicate the desirability of providing a roadside clear zone or a guardrail can include reported crashes or evidence of roadside encroachments. However, both roadside encroachments and crashes are generally rare on low-volume roads.

4.8 PEDESTRIAN AND BICYCLE FACILITIES

The needs of pedestrians and bicyclists should be assessed during the design or improvement of low-volume roads. Low-volume roads are typically sparsely developed; however, as these roads transition to or connect with developing corridors, the transportation needs for all users (motorists, bicyclists, transit riders, and pedestrians) should be investigated through collaboration with the nearby community and local and regional planning agencies.

Many rural low-volume roads have little pedestrian activity, and facilities specifically intended for pedestrians may not be needed. Pedestrian activity levels may be greater on urban low-volume roads and streets and, where appropriate given pedestrian volumes and the character of surrounding development, provision of pedestrian facilities may be considered. Where pedestrian facilities are provided, they must be accessible to and usable by individuals with disabilities (**6, 16**). Design of pedestrian facilities should be guided by local policies and the AASHTO *Guide for Planning, Operation, and Design of Pedestrian Facilities* (**1**). Further guidance on the design of accessible pedestrian facilities is presented in the *Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way* (**15**).

Bicycle volumes vary widely on low-volume roads. Many low-volume roads have little or no bicycle activity. However, some low-volume roads in both rural and urban areas may provide bicyclists with excellent alternatives to higher volume roads and may be designated as bicycle routes. Some recreational and scenic roads may serve bicyclists, as well as motor vehicles, attracted by the natural beauty of the region. Low-volume roads are unlikely to need designated bicycle lanes or separated bicycle paths, but paved shoulders are useful to bicyclists and may be considered, particularly on major access routes with higher traffic volumes. The AASHTO *Guide for the Development of Bicycle Facilities* (**4**) provides guidance on choosing appropriate facility types and design elements.

4.9 UNPAVED ROADS

Many low-volume roads have unpaved surfaces. Unpaved roads are generally appropriate for all functional subclasses of low-volume roads. Major access roads often have paved surfaces because they serve higher traffic volumes, but this is not considered mandatory. In particular, resource recovery (e.g., logging) roads and agricultural access roads in rural areas are frequently unpaved. Provision of an unpaved surface is an economic decision that is appropriate for many low-volume roads for which the cost of constructing and maintaining a paved surface would be prohibitive.

The likelihood of crashes on unpaved roads has been addressed in NCHRP Report 362 (**20**). This research established that crash rates are generally higher for unpaved roads than for paved roads for design volumes of 250 vehicles per day or more. The risk assessment by Neuman (**11**) found that roads in rural areas generally reach the threshold at which paving the road would be expected to result in one less severe crash every 10 to 15 years in the traffic volume range between 300 to 350 vehicles per day. However, there are no specific guidelines that indicate the maximum traffic volume level for which unpaved surfaces are appropriate.

NCHRP Report 362 (**20**) found crash rates for unpaved roads to be lower for narrower roadway widths. Therefore, existing unpaved roads should not generally be widened as a crash-reduction measure unless there is evidence of a site-specific crash pattern that may be corrected by widening.

Unpaved roads are intended to operate at low to moderate speeds. Design speeds for unpaved roads should normally be 45 mph [70 km/h] or less, but may occasionally be as high as 50 mph [80 km/h] in situations the designer considers appropriate.

Provision of roadside clear zones, flatter slopes, or traffic barriers is generally inconsistent with the economic decision to build and maintain an unpaved surface and is not generally needed for the low-speed environment of an unpaved road.

Design of horizontal alignment on unpaved roads differs from paved roads because paved and unpaved roads have different surface friction characteristics and because unpaved roads are typically designed for low-speed operation.

Table 4-12 presents guidelines for the minimum radius of curvature for unpaved surfaces with no superelevation for application on low-volume roads. The table is based on the design criteria of the United States Forest Service (**17**), which operates many unpaved roads. The minimum radius of curvature is a function of traction coefficient, which in turn is a function of the surface type (earth, gravel, crushed rock, packed snow, etc.) and the surface condition (dry, wet, ice, etc.) as shown in Table 4-13. The recommended minimum curve radii in Table 4-12 are based on Equation 4-9 using a side friction factor, *f*, that is 50 percent of the specified traction coefficient shown in Table 4-13. Use of high values of friction coefficient for design allows the designer to select smaller curve radii than would otherwise be used. Of course, the selection of a high traction coefficient is consistent with a higher surface type or with an assumption that poor surface conditions such as snow, ice, or wet pavement, or both, are not sufficiently frequent for use as a design control. The choice of the appropriate surface condition from Table 4-13 should be based on the engineering judgment of the designer based on site-specific conditions.

Smaller curve radii than those shown in Table 4-12 may be used where superelevation is provided. The minimum radius of curvature for such cases can be determined with Equation 4-2.

When an existing unpaved road is to be paved, a review of all geometric design elements of the road should be undertaken to assess their suitability for the higher speeds that are likely on a paved road.

Table 4-12. Guidelines for Minimum Radius of Curvature for New Construction of Unpaved Surfaces with No Superelevation [adapted from (17)]

U.S. Customary					
	Minimum Radius (ft)				
	Traction Coefficient				
Design Speed (mph)	0.7	0.6	0.5	0.4	0.3
15	50	50	60	75	100
20	75	90	110	135	180
25	120	140	170	210	280
30	170	200	240	300	400
35	235	275	330	410	545
40	305	360	430	535	715
45	390	450	540	675	900

Metric					
	Minimum Radius (m)				
	Traction Coefficient				
Design Speed (km/h)	0.7	0.6	0.5	0.4	0.3
20	15	15	15	20	25
30	20	25	30	35	50
40	40	45	50	65	85
50	60	70	80	100	135
60	85	95	115	145	190
70	110	130	155	195	260

U.S. Customary	Metric	
$$R_{\min} = \frac{V^2}{15(f_{\max})}$$	$$R_{\min} = \frac{V^2}{127(f_{\max})}$$	(4-9)
Where: R_{min} = Minimum curve radius (ft) of curvature for surfaces with no superelevation V = vehicle speed (mph) f_{max} = (0.50) Traction Coefficient	Where: R_{min} = Minimum curve radius (m) of curvature for surfaces with no superelevation V = vehicle speed (km/h) f_{max} = (0.50) Traction Coefficient	

Table 4-13. Traction Coefficients Used in Design of Horizontal Alignment on Unpaved Roads (17)

Material	Surface Condition	
	Dry	Wet
Gravel, packed, oiled	0.50–0.85	0.40–0.80
Gravel, loose	0.40–0.70	0.36–0.75
Rock, crushed	0.55–0.75	0.55–0.75
Wet earth	0.55–0.65	0.40–0.50
Dry-packed snow	0.20–0.55	—
Loosely packed snow	0.10–0.60	—
Snow, lightly sanded	0.29–0.31	—
Snow, lightly sanded with chains in use	0.34	—

4.10 TWO-WAY SINGLE-LANE ROADS

Two-way single-lane roads may be used in constrained locations, where traffic volumes are extremely low. The U.S. Forest Service *Road Preconstruction Handbook* (**17**), which deals primarily with rural recreational and scenic roads and rural resource recovery roads, recommends that single-lane roads be used when the estimated design volume is less than 100 vehicles per day, and that two-lane roads be considered for roads with design volumes above 100 vehicles per day. For other roadway types, single-lane cross sections are normally used on local roads where design volumes are less than 50 vehicles per day. In addition to ADT thresholds, other considerations for choosing a single-lane or double-lane cross section include the characteristics of the vehicles and drivers who will be using the road. On resource recovery roads used by professional drivers who are often in contact with one another by radio, two-way single-lane roads may be more appropriate than on roads used by unfamiliar drivers or by large vehicles (such as tour buses). Two-way single-lane roads are designed to operate at low speeds, typically no more than 40 mph [60 km/h].

Two-way single-lane roads are often unpaved and normally have widths in the range from 10 to 14 ft [3.0 to 4.3 m], depending on design speed and design vehicle. Single-lane roads narrower than 10 ft [3.0 m] or wider than 14 ft [4.3 m] are not recommended. Curve widening may be needed in some locations to provide for offtracking of tractor–trailers and some vehicle–trailer combinations. Design values of stopping sight distance for two-way single-lane roads should be twice the stopping sight distance for a comparable two-lane road, as shown in Table 4-7. USFS guidelines recommend that turnouts be provided at regular intervals on two-way single-lane roads to allow opposing vehicles to pass one another. The location of turnouts should consider topography and horizontal and vertical alignment. In some cases, particularly where increased sight distances are impractical, widening of the roadway at crests should be considered. Refer to the U.S. Forest Service *Road Preconstruction Handbook* (**17**) for additional design guidance for two-way single-lane roads.

5 Design Examples

This chapter presents ten examples of the application of the design guidelines for low-volume roads presented in Chapter 4. The ten examples are hypothetical situations that illustrate how the design guidelines are intended to be applied. The general subjects of the ten examples are:

1. New construction of a major access road in a rural area,
2. Resurfacing of an existing major access road in a rural area,
3. Rehabilitation of a minor collector road in a rural area,
4. New construction of an industrial or commercial access road in a rural area,
5. Reconstruction of a minor access road in a rural area,
6. New construction of an unpaved minor access road in a rural area,
7. New construction of an urban residential street,
8. Reconstruction of an urban industrial or commercial access street,
9. Rehabilitation of a rural recreational or scenic road, and
10. Resurfacing of an urban major access road.

The specific examples are presented below.

5.1 EXAMPLE 1—NEW CONSTRUCTION OF A MAJOR ACCESS ROAD IN A RURAL AREA

A county engineer has been given the job of designing a new rural major access road with a design speed of 50 mph [80 km/h]. The road is functionally classified as a local road and will provide access to adjoining property as well as to several minor access roads. The roadway will be located in rolling terrain and initial traffic volumes are expected to be in the range of 275 to 300 vehicles per day. The design year is 20 years into the future, by which time the traffic volumes are expected to grow to no more than 350 vehicles per day. Thus, the engineer has concluded that it is appropriate to use the design guidelines for low-volume roads presented in Chapter 4.

In summary, the traffic engineer knows the following information before the design process begins:

Project Type:	new construction
Area Type:	rural
Functional Subclass:	rural major access road
Design Speed:	50 mph [80 km/h]
Design Traffic Volume:	350 vehicles per day

Cross Section

Guidelines for total roadway width for low-volume roads in rural areas are presented in Table 4-1. Since the new roadway will be a major access road with a design speed of 50 mph [80 km/h], the county engineer selects from Table 4-1 a total roadway width of 20.0 ft [6.0 m] for the project.

The engineer determines that the project could be built within a minimum right-of-way width of 48 ft [15 m]. However, because right-of-way can be easily acquired for this facility on a new alignment, the engineer chooses a more generous right-of-way width of 60 ft [18 m]. This provides flexibility to accommodate future increases in traffic volume and future widening needs that are not currently anticipated but could occur.

Horizontal Alignment

Maximum Friction Factor and Minimum Radius

Table 4-3 presents the values of f_{max} and R_{min} used in the design of higher volume roadways. However, for the design of low-volume roads without substantial truck volumes, acceptable operations can be obtained with smaller curve radii than those shown in Table 4-3. Table 4-5 presents values of f_{max} and R_{min} that can be used in preference to those in Table 4-3 in the design of low-volume roads with design volumes of 251 to 400 vehicles per day and limited truck volumes.

The county in which the project will be constructed uses a maximum superelevation rate, e_{max}, of 8 percent. Thus, the county engineer should confirm that all curves in the horizontal alignment have a minimum radius of 760 ft [230 m] if the design is based on Table 4-3 or a minimum radius of 585 ft [140 m] if the design is based on Table 4-5. The county engineer, however, is not faced with any real physical constraints in connection with this project. That is, the county has purchased plenty of right-of-way, there are no environmentally sensitive areas to be avoided, and there are no physical constraints or adjacent development that influence the design. Therefore, the county engineer is able to design the horizontal alignment using the guidelines in Table 4-3 which are based on the AASHTO Green Book (**5**).

Superelevation

For each individual horizontal curve, the designer selects the design superelevation based on the criteria in Chapter 3 of the AASHTO Green Book (5), based on a design speed of 50 mph [80 km/h] and a maximum superelevation rate of 8 percent.

Superelevation Transition

The county engineer designs the superelevation transitions in accordance with the criteria presented in Chapter 3 of the AASHTO Green Book (5).

Stopping Sight Distance

Design Stopping Sight Distance

Table 4-7 presents the design stopping sight distance criteria for low-volume roads. Since the new roadway will have a design speed of 50 mph [80 km/h] and a projected traffic volume of 350 vehicles per day, the minimum stopping sight distance for this project is 350 ft [110 m].

Crest Vertical Curves

Table 4-9 shows that to achieve the design stopping sight distance of 350 ft [110 m], all crest vertical curves should be designed with a rate of vertical curvature, *K*, of at least 57 ft [19 m] per percent difference in grade.

Sag Vertical Curves

There are no special guidelines for design of sag vertical curves on low-volume roads. Therefore, the county engineer should design the sag vertical curves in accordance with Chapter 5 of the AASHTO Green Book (5).

Horizontal Curves

The county engineer uses Equation 4-4 or Table 4-8 to determine the width that should be clear of sight obstructions on the inside of each horizontal curve. The horizontal sightline offset, *HSO*, computed with Equation 4-4 or determined from Table 4-8 is measured from the center of the inside lane.

Intersection Sight Distance

All intersections on the new major access road will have stop control on the intersecting crossroad. Therefore, no approach sight triangles are needed on the crossroad approaches.

Departure sight triangles, like those shown in Figure 4-3B, should be provided for each crossroad approach. Due to the rolling terrain, the legs of the sight triangles located along the major road may not be equal to the full intersection sight distance for stop-controlled intersections, as presented in Chapter 9 of the AASHTO Green Book (**5**). In constrained situations, the length of the leg of the departure triangles along the major road will be at least equal to the stopping sight distance of 350 ft [110 m], as determined from Table 4-7. These departure sight triangles not only allow drivers of crossroad vehicles to see major-road traffic before they begin to enter the major road, they also allow major-road drivers to see vehicles on the crossroad approach.

Roadside Design

Clear Zone Width

While no specific minimum clear zone width is needed, the county engineer has found that a clear zone width of 6.5 ft [2 m] can be provided at little or no additional cost. Therefore, a clear zone of this width will be provided. Since the horizontal alignment will be designed according to the AASHTO Green Book (**5**), the county engineer determined that there was no need to provide a widened clear zone on the outside of horizontal curves.

Traffic Barriers

The engineer has found no locations within the project where guardrail or other traffic barriers are needed. Therefore, no barriers are included in the design.

Other Design Features

All other geometric design elements will be provided in accordance with Chapter 5 of the AASHTO Green Book (**5**).

5.2 EXAMPLE 2—RESURFACING OF AN EXISTING MAJOR ACCESS ROAD IN A RURAL AREA

A state highway agency is about to begin a resurfacing project on a major access road with a design speed of 55 mph [90 km/h]. The road is functionally classified as a local road and provides access to adjoining property as well as to several minor access roads. The roadway is located in level terrain and carries traffic volumes in the range of 150 to 175 vehicles per day. Little traffic volume growth is expected; the traffic volume in the design year, 20 years from now, is not expected to exceed 200 vehicles per day. Because of the functional classification of the road and its low volumes, the highway agency has concluded that it is appropriate to use the design guidelines for low-volume roads presented in Chapter 4.

One of the horizontal curves within the project has experienced a site-specific crash pattern—two single-vehicle crashes in which a vehicle ran off the outside of the curve have occurred in the last seven years, and several skid marks near the same site have been noted in the field as well. Therefore, the state highway agency has decided to incorporate improvements to the horizontal curve as part of the planned resurfacing project.

In summary, the traffic engineer responsible for this state project knows the following information before the resurfacing process begins:

Project Type:	resurfacing of an existing road
Area Type:	rural
Functional Subclass:	rural major access road
Design Speed:	55 mph [90 km/h]
Design Traffic Volume:	200 vehicles per day

Cross Section

The existing total roadway width within the project is 20.7 ft [6.3 m]. The cross-section guidelines for new construction for a rural major access road with a design speed of 55 mph [90 km/h] in Table 4-1 indicate that a total roadway width of 22.0 ft [6.6 m] would be appropriate. However, because no site-specific crash patterns attributable to cross section width have been found at the site, the existing total roadway width of 20.7 ft [6.3 m] may remain in place.

Horizontal Alignment

For improvement projects to existing roads, the guidelines suggest that for curves on low-volume roads with higher speeds, reconstruction without changing the existing curve geometry and cross section or making other improvements is acceptable if:

1. The nominal design speed of the curve is within 10 mph or 20 km/h of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific crash pattern associated with the curve.

The guidelines also suggest that even with such evidence, curve improvements should focus on low-cost measures designed to control speeds, enhance curve tracking, or mitigate roadside encroachment severity. Therefore, the engineer decides to address the crash patterns at the horizontal curve with more cost-effective solutions than curve flattening and reconstruction.

Since both the crash history and the skid marks at the horizontal curve location noted above are an indication of excessive speed, the engineer recommends implementing measures to reduce vehicle speeds on the curve. Specifically, the state highway agency will place curve warning signs in advance of the curve and improve the pavement markings throughout the curve.

Stopping Sight Distance

There is no evidence of a site-specific crash pattern related to stopping sight distance. Therefore, no modifications will be made to the horizontal and vertical alignments.

Intersection Sight Distance

There is no evidence of a site-specific crash pattern related to intersection sight distance. Therefore, no modifications will be made to increase intersection sight distance at any of the intersections along the resurfacing project.

Roadside Design

There is no evidence of a site-specific crash pattern indicating the desirability of providing a wider roadside clear zone or guardrail. Therefore, no improvements will be made to the roadside design.

5.3 EXAMPLE 3—REHABILITATION OF A MINOR COLLECTOR ROAD IN A RURAL AREA

A state highway agency plans to rehabilitate a minor collector road with a design speed of 60 mph [100 km/h] in a rural area. This minor collector road is part of the state primary road system (i.e., a numbered route), but it serves a traffic volume of only 300 vehicles per day. Most drivers on this road use it at least weekly for access from two small villages to the county seat where stores and services are available. The population of the area is declining; traffic volumes have decreased over the last 10 years and are expected to continue to decrease. Therefore, the design guidelines in Chapter 4 are applicable to this road and it is treated for purposes of the guidelines as a rural major access road. It should be noted that the designation of this road as a state numbered route has no bearing on its classification for application of the guidelines. The road should be treated in the same manner under these guidelines whether it is under state, county, or township jurisdiction.

In summary, the design engineer responsible for this state project knows the following information as planning for the project begins:

Project Type:	rehabilitation of an existing road
Area Type:	rural
Functional Subclass:	rural major access road
Design Speed:	60 mph [100 km/h]
Design Traffic Volume:	300 vehicles per day

Cross Section

The existing total roadway width within the project over most of its length is 24 ft [7.2 m]. This exceeds the roadway width guidelines of 22 ft [6.6 m] for major access roads shown in Table 4-1. Furthermore, there is no evidence of any site-specific crash pattern related to roadway width. Therefore, the existing roadway width may remain in place.

One 2-mi [3.2-km] section of the project has a total roadway width of 20 ft [6.1 m]. While there is no evidence of an existing crash pattern that would make widening desirable, the design engineer decides that this section should be widened to a total roadway width of 24 ft [7.2 m] for consistency with the rest of the project.

Horizontal Alignment

For improvement projects to existing roads, the guidelines suggest that, for curves with higher speeds, there is no need to change the existing curve geometry and cross section or to make other improvements if:

1. The nominal design speed of the curve is within 10 mph or 20 km/h of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific crash pattern associated with the curve.

All horizontal curves on the project were found to meet these criteria.

Stopping Sight Distance

There is no evidence of a site-specific crash pattern related to stopping sight distance. Therefore, no modifications will be made to the horizontal and vertical alignments.

Intersection Sight Distance

There is no evidence of a site-specific crash pattern related to intersection sight distance. Therefore, no modifications will be made to increase intersection sight distance at any of the intersections along the rehabilitation project.

Roadside Design

There is no evidence of a site-specific crash pattern indicating the desirability of providing a wider roadside clear zone or guardrail. Therefore, no improvements will be made to roadside design.

5.4 EXAMPLE 4—NEW CONSTRUCTION OF AN INDUSTRIAL OR COMMERCIAL ACCESS ROAD IN A RURAL AREA

An engineering consultant has been hired by a township to design a new rural industrial or commercial access road with a design speed of 30 mph [50 km/h]. The road is functionally classified as a local road and will function solely to provide access to adjoining property. The roadway will be located in level terrain and initial traffic volumes are expected to be around 80 vehicles per day. The design year is 20 years into the future, by which time the traffic volumes are expected to grow to no more than 100 vehicles per day. Thus, the engineer has concluded that it is appropriate to use the design guidelines for low-volume roads presented in Chapter 4.

In summary, the consultant knows the following information before the design process begins:

Project Type:	new construction
Area Type:	rural
Functional Subclass:	rural industrial or commercial access road
Design Speed:	30 mph [50 km/h]
Design Traffic Volume:	100 vehicles per day

Cross Section

Guidelines for total roadway width for low-volume roads in rural areas are presented in Table 4-1. Since the new roadway will be an industrial or commercial access road with a design speed of 30 mph [50 km/h], the consultant selects from Table 4-1 a total roadway width of 22.5 ft [6.8 m] for the project.

Horizontal Alignment

Maximum Friction Factor and Minimum Radius

Table 4-3 presents the values of f_{max} and R_{min} used in design of higher volume roadways. However, for the design of low-volume roads, even those with substantial proportions of truck traffic, acceptable operations can be obtained with smaller curve radii than those shown in Table 4-3. Table 4-6 presents values of f_{max} and R_{min} that can be used in preference to those in Table 4-3 in the design of low-volume roads with substantial proportions of truck traffic.

The township in which the project will be constructed uses a maximum superelevation rate, e_{max}, of 6 percent. Therefore, the consulting engineer should confirm that all curves in the horizontal alignment have a minimum radius of 230 ft [80 m] if the design is based on Table 4-3 or a minimum radius of 145 ft [60 m] if the design is based on Table 4-6. The roadway alignment is constrained by the presence of existing structures on private property and environmentally sensitive wetlands which can be avoided if curve radii based on Table 4-6 are used. Therefore, the consulting engineer decides that the horizontal alignment should be designed on the basis of the values of f_{max} and R_{min} shown in Table 4-6.

Superelevation

For each individual horizontal curve, the designer selects the design superelevation based on the criteria in Chapter 3 of the AASHTO Green Book (5) based on a design speed of 30 mph [50 km/h] and a maximum superelevation rate of 6 percent.

Superelevation Transition

The consultant designs the superelevation transitions in accordance with the criteria presented in Chapter 3 of the AASHTO Green Book (5).

Stopping Sight Distance

Design Stopping Sight Distance

Table 4-7 presents the design stopping sight distance criteria for low-volume roads. Since the new roadway will have a design speed of 30 mph [50 km/h] and a projected traffic volume of 100 vehicles per day, the minimum stopping sight distance for this project is 135 ft [45 m].

Crest Vertical Curves

Table 4-9 shows that to achieve the design stopping sight distance of 135 ft [45 m], all crest vertical curves should be designed with a rate of vertical curvature, *K*, of at least 9 ft [4 m] per percent difference in grade.

Sag Vertical Curves

There are no special guidelines for design of sag vertical curves on low-volume roads. Therefore, the consultant designs the sag vertical curves in accordance with Chapter 5 of the AASHTO Green Book (5).

Horizontal Curves

The consultant uses Equation 4-4 to determine the width that should be clear of sight obstructions on the inside of each horizontal curve. The horizontal sightline offset, *HSO*, computed with Equation 4-4 or determined from Table 4-8 is measured from the center of the inside lane.

Intersection Sight Distance

Some intersections on the new minor access road will have stop control on the intersecting minor access roads. Only departure sight triangles are needed for these intersections. The remaining intersections will have no control on the intersecting roads. Only approach sight triangles are needed at these uncontrolled intersections.

Approach Sight Triangles

Approach sight triangles, like those shown in Figure 4-3A, should be provided for each approach to each of the uncontrolled intersections. The consultant selects values for the legs of the approach sight triangle from Table 4-10. The leg extending along the road being constructed, with a 30 mph [50 km/h] design speed, should be at least 120 ft [40 m]. The leg extending along the intersecting roadway will be determined from Table 4-10 based on the design speed of that roadway.

Departure Sight Triangle

Departure sight triangles, like those shown in Figure 4-3B, should be provided for each minor-road approach to each of the stop-controlled intersections. The length of the leg of the departure triangle along the major road will be at least equal the stopping sight distance of 135 ft [45 m], as determined from Table 4-7. The length of the departure sight triangle along the crossroad approach should be 14.4 ft [4.4 m].

Roadside Design

Clear Zone Width

Since no specific minimum clear zone width is required, and there are both right-of-way and environmental constraints, the consultant does not provide any clear zone on this project.

Traffic Barriers

The engineer has found no locations within the project where guardrail or other traffic barriers are needed. Therefore, no barriers are included in the design.

Other Design Features

All other geometric design elements will be provided in accordance with Chapter 5 of the AASHTO Green Book (**5**).

5.5 EXAMPLE 5—RECONSTRUCTION OF A MINOR ACCESS ROAD IN A RURAL AREA

A rural county is about to begin a reconstruction project on a minor access road with a design speed of 25 mph [40 km/h]. The road is functionally classified as a local road and provides access to adjoining property. The traffic volume on this road, in the range of 100 to 125 vehicles per day, has been declining slightly in recent years and is expected to continue to decline over the 20-year design period.

The roadway pavement has failed and the reconstruction project will therefore involve a total replacement of the pavement structure down to the subgrade. The county engineer is responsible for determining any geometric improvements that should be made in conjunction with the reconstruction project.

In summary, the county engineer knows the following information before the resurfacing process begins:

Project Type:	reconstruction of an existing roadway
Area Type:	rural
Functional Subclass:	rural minor access road
Design Speed:	25 mph [40 km/h]
Design Traffic Volume:	100 to 125 vehicles per day or less

Cross Section

There is no evidence of any site-specific crash pattern. Therefore, in accordance with the guidelines for existing roadways, the county engineer determines that there is no need to modify the cross-section width of the existing roadway.

Horizontal Alignment

Maximum Friction Factor and Minimum Radius

For improvement projects, the guidelines suggest that for curves on lower speed low-volume roads, reconstruction without changing the existing curve geometry and cross section is acceptable if:

1. The nominal design speed of the curve is within 20 mph or 30 km/h of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific crash pattern associated with the curve.

The county engineer determines that both of these guidelines are met. Therefore, he or she concludes that no improvements to the horizontal alignment are needed.

Stopping Sight Distance

There is no evidence of a site-specific crash pattern attributable to inadequate sight distance. Therefore, no modifications will be made to the horizontal and vertical alignments.

Intersection Sight Distance

There is no evidence of a site-specific crash pattern related to intersection sight distance. Therefore, no modifications will be made to increase intersection sight distance at any of the intersections along the reconstruction project.

Roadside Design

There is no evidence of a site-specific crash pattern indicating the desirability of providing a roadside clear zone or a guardrail. There is a line of attractive 100-year-old trees along both sides of a long tangent segment of the roadway. Removal of these trees would bring strong objections from local residents and there is no evidence that, given the low traffic volumes on the roadway, vehicles are likely to run off the road and strike these trees. Therefore, the county engineer decides that these trees should remain in place. However, the county engineer does find that 16 ft [5 m] clear zones can be provided on the outside of two horizontal curves at little or no additional cost, so a decision to provide these clear zones is made.

5.6 EXAMPLE 6—NEW CONSTRUCTION OF AN UNPAVED MINOR ACCESS ROAD IN A RURAL AREA

A rural township is planning to construct an unpaved rural minor access road on a new alignment. The design speed will be 40 mph [60 km/h] and the traffic volume on the road is expected to be 75 vehicles per day initially and 90 vehicles per day after 20 years. Therefore, the consulting engineer engaged by the township has determined that it is appropriate to apply the design guidelines presented in Chapter 4 to this project.

In summary, the consulting engineer has the following information about the project:

Project Type:	new construction of an unpaved road
Area Type:	rural
Functional Subclass:	rural minor access road
Design Speed:	40 mph [60 km/h]
Design Traffic Volume:	90 vehicles per day

Cross Section

The total roadway width selected for the road is 18 ft [5.4 m] based on the guidelines presented in Table 4-1.

Horizontal Alignment

The surfacing material selected for the roadway is loose gravel with an expected traction coefficient of 0.5 under wet conditions, which is consistent with Table 4-13. A traction coefficient of 0.5 corresponds to a side friction factor, *f*, of 0.25. Table 4-12 indicates that the appropriate minimum radius of curvature for a design speed of 40 mph [60 km/h] and a traction coefficient of 0.5 is 430 ft [115 m]. This minimum radius applies to curves with no superelevation. If superelevation of 4 percent is provided, the equivalent minimum radius, determined from Equation 4-2, curves could be built with a minimum radius of 370 ft [100 m]. In fact, the sharpest curve designed by the consultant for the project has a radius of 492 ft [150 m].

Stopping Sight Distance

Design Stopping Sight Distance

Table 4-7 presents the design stopping sight distance guidelines for low-volume roads. Since the roadway will have a design speed of 40 mph [60 km/h] and a design traffic volume under 100 vehicles per day, the minimum stopping sight distance for this roadway should be 215 ft [60 m].

Crest Vertical Curves

Table 4-9 shows that to achieve the design stopping sight distance of 215 ft [60 m], all crest vertical curves should be designed with a rate of vertical curvature, *K*, of at least 22 ft [6 m] per percent difference in grade.

Sag Vertical Curves

There are no special guidelines for sag vertical curves on low-volume roads. Therefore, the engineer designs the sag vertical curves in accordance with Chapter 5 of the AASHTO Green Book (5).

Horizontal Curves

The engineer should use Equation 4-4 on Table 4-8 to determine the width that should be clear of sight obstructions on the inside of each horizontal curve. The horizontal sightline offset, *HSO*, computed with Equation 4-4 as determined from Table 4-8 is normally measured from the center of the inside lane. Since lanes are not marked on an unpaved road, the clear sight width on the 18-ft [5.4-m] roadway should be measured from a point in the roadway 4.5 ft [1.35 m] from its inside edge.

Intersection Sight Distance

There are only two intersections on the new minor access road. The first is a four-leg uncontrolled intersection with another unpaved roadway that has a design traffic volume of 30 vehicles per day. The clear sight triangles for this intersection are determined from Table 4-10. The second intersection is a three-leg intersection where the new minor access road terminates with stop-control at an existing collector road with a design traffic volume of 900 vehicles per day. Because this intersection has two legs whose traffic volumes exceed 400 vehicles per day, the design guidelines in Chapter 4 do not apply.

The clear sight triangles for this intersection should be determined in accordance with Chapter 9 of the AASHTO Green Book (**5**).

Roadside Design

Clear Zone Width

While no specific minimum clear zone is needed, the engineer has found that a 6.5-ft [2-m] clear zone can be provided throughout the project's length at little or no additional cost, because all roadside obstacles within that area would normally be removed during construction.

Traffic Barriers

No need for guardrail or other traffic barriers has been identified. Therefore, no barriers are included in the design.

Other Design Features

All other geometric design elements will be provided in accordance with Chapter 5 of the AASHTO Green Book (**5**).

5.7 EXAMPLE 7—NEW CONSTRUCTION OF AN URBAN RESIDENTIAL STREET

A city traffic engineer has been given the job of reviewing the plans prepared by a developer for a new residential street on which a new housing development is planned. The road is functionally classified as an urban residential street and will serve to provide access solely to single-family residences. The street will have a design speed of 30 mph [50 km/h] and is expected to carry traffic volumes in the range of 85 to 100 vehicles per day. The design year is 20 years into the future, by which time the traffic volumes are expected to grow to no more than 150 vehicles per day. Thus, the engineer has concluded that it is appropriate to use the design guidelines for low-volume roads presented in Chapter 4.

In summary, the traffic engineer knows the following information before the design review begins:

Project Type:	new construction
Area Type:	urban
Functional Subclass:	urban residential street
Design Speed:	30 mph [50 km/h]
Design Traffic Volume:	150 vehicles per day

Cross Section

Guidelines for total roadway width for urban residential streets are presented in Table 4-2. These widths incorporate consideration of access for fire trucks and other emergency vehicles. While there will not be any marked parking spaces on the proposed residential street, parking will be permitted on both sides of the street. The suburban character of the development represents a medium development density. Therefore, the traffic engineer determines from Table 4-2 that a total roadway width of 26 ft [7.9 m] is appropriate for the project. The developer has recommended a roadway width of 25 ft [7.6 m], so the engineer requests that this be increased to 26 ft [7.9 m].

Horizontal Alignment

Maximum Friction Factor and Minimum Radius

Table 4-3 presents the values of f_{max} and R_{min} used in the design of higher volume roadways. Urban residential streets with average daily traffic volumes of 400 vehicles per day or less should be designed in accordance with the limiting values of f_{max} and R_{min} presented in Table 4-3, whenever practical.

If constrained conditions were present, the design could use a minimum radius of curvature of 85 ft [35 m] with the city's maximum superelevation rate, e_{max}, of 4 percent, based on Table 4-4. However, since there are no structures currently present in the right-of-way and there are no other physical constraints, the horizontal alignment can be designed in accordance with the guidelines in Table 4-3 which are based on the AASHTO Green Book (5). Therefore, the city engineer selects from Table 4-3 a minimum radius of 250 ft [85 m], corresponding to a design speed of 20 mph [50 km/h] and the city's maximum superelevation rate, e_{max}, of 4 percent. For the one horizontal curve on the project, the developer has chosen a radius of 724 ft [221 m]. The engineer concludes that the proposed horizontal curve design is acceptable.

Superelevation

For the one horizontal curve on the project, the designer selects the design superelevation based on the criteria in Chapter 3 of the AASHTO Green Book (5) for a design speed of 30 mph [50 km/h] and a maximum superelevation rate of 4 percent.

Superelevation Transition

The city traffic engineer designs the superelevation transitions in accordance with the criteria presented in Chapter 3 of the AASHTO Green Book (5).

Stopping Sight Distance

Design Stopping Sight Distance

Table 4-7 presents the design stopping sight distance guidelines for low-volume roads. Since the new roadway will have a design speed of 30 mph [50 km/h] and a projected traffic volume of 150 vehicles per day, the minimum stopping sight distance for this project should be 165 ft [55 m] near intersections and 135 ft [45 m] away from intersections.

Crest Vertical Curves

Table 4-9 shows that to achieve the design stopping sight distance of 135 ft [45 m], all crest vertical curves should be designed with a rate of vertical curvature, *K*, of at least 9 ft [4 m] per percent difference in grade. The engineer finds that both crest vertical curves on the project have been designed appropriately.

Sag Vertical Curves

There are no special guidelines for design of sag vertical curves on low-volume roads. Therefore, the city traffic engineer concludes that the sag vertical curves on the project should be designed in accordance with Chapter 5 of the AASHTO Green Book (**5**).

Horizontal Curves

The city traffic engineer uses Equation 4-4 to determine the width that should be clear of sight obstructions on the inside of each horizontal curve. The horizontal sightline offset, *HSO*, computed with Equation 4-4 or determined from Table 4-8 is measured from the center of the inside lane. After applying this criterion, the city traffic engineer finds that a decorative sculpture planned for placement by the developer constitutes a horizontal sight obstruction. Based on the engineer's recommendation, the sculpture is moved to an alternative location.

Intersection Sight Distance

All intersections on the new urban residential street will have stop control on the intersecting crossroads. Therefore, no approach sight triangles are needed on these crossroads.

Departure sight triangles, like those shown in Figure 4-3B, should be provided for each crossroad approach. The legs of the sight triangles located along the urban residential street should be equal to the full intersection sight distance for stop-controlled intersections, as presented in Chapter 9 of the AASHTO Green Book (**5**).

Roadside Design

Clear Zone Width

Since no specific minimum clear zone width is required, no clear zone is provided on this project.

Traffic Barriers

The city traffic engineer has found no locations within the project where guardrail or other traffic barriers are needed. Therefore, no barriers are included in the design.

Other Design Features

All other geometric design elements will be provided in accordance with Chapter 5 of the AASHTO Green Book (**5**).

5.8 EXAMPLE 8—RECONSTRUCTION OF AN URBAN INDUSTRIAL OR COMMERCIAL ACCESS STREET

A city is about to reconstruct a short unpaved access street. A member of the city engineering staff has been given the responsibility to determine what geometric improvements should be made in conjunction with the reconstruction project. The road is functionally classified as an urban industrial or commercial access street and serves a paper factory that generates a substantial volume of truck and heavy vehicle trips. The primary function of the street is to provide access from the factory to the local highway network. The access street has a design speed of 35 mph [60 km/h]. The street carries traffic volumes in the range of 175 to 200 vehicles per day. Over the 20-year design period, the traffic volume is expected to grow to 225 to 250 vehicles per day. The reconstruction project will involve paving the street.

In summary, the city engineer knows the following information before the resurfacing process begins:

Project Type:	reconstruction of an existing street
Area Type:	urban
Functional Subclass:	urban industrial or commercial access street
Design Speed:	35 mph [60 km/h]
Design Traffic Volume:	225 to 250 vehicles per day

Cross Section

The existing unpaved street width is 22.5 ft [6.8 m], which is equal to the recommended cross section width for an industrial or commercial access street with a 35 mph [60 km/h] design speed as shown in Table 4-1.

There is no evidence of any site-specific crash patterns on the existing street. Therefore, in accordance with the guidelines for existing roadways, the county engineer determines that there is no need to modify the cross-section width of the existing roadway.

Horizontal Alignment

Maximum Friction Factor and Minimum Radius

For improvement projects, the guidelines suggest that for curves on lower speed low-volume roads, reconstruction without changing the existing curve geometry and cross section is acceptable if:

1. The nominal design speed of the curve is within 20 mph or 30 km/h of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific crash pattern associated with the curve.

The city engineer determines that both criteria are met. Therefore, he or she is justified in not making any improvements to the horizontal alignment.

Stopping Sight Distance

There is no evidence of a site-specific crash pattern attributable to inadequate sight distance. Therefore, no modifications will be made to the horizontal and vertical alignments.

Intersection Sight Distance

There is no evidence of a site-specific crash pattern related to intersection sight distance. Therefore, no modifications will be made to increase intersection sight distance at any of the intersections along the resurfacing project.

Roadside Design

There is no evidence of a site-specific crash pattern indicating the desirability of providing a roadside clear zone or a guardrail. Therefore, no improvements will be made to the roadside design.

5.9 EXAMPLE 9—REHABILITATION OF A RURAL RECREATIONAL OR SCENIC ROAD

A state DOT has plans to rehabilitate a rural state route that connects a primary state highway to the entrance of a state park. The road provides a scenic drive along a river that is used for canoeing and fishing, and provides access to campgrounds and rental cabins. When the roadway was designed, it carried 300 vehicles per day and was considered a rural recreational or scenic road. Today, during the peak months (April through October), the average daily traffic volume is approximately 1,200 vehicles per day and is expected to remain at that level for the roadway's design life. The road has been reclassified as a minor collector. There is some limited bicycle flow to and from the state park during the months of July and August. The design speed of the road is 45 mph [70 km/h]. Thus, the engineer has concluded that it is appropriate to use the design guidelines for low-volume roads presented in Chapter 4.

In summary, the traffic engineer knows the following information before the design process begins:

Project Type:	rehabilitation of existing roadway
Area Type:	rural
Functional Subclass:	formerly, rural scenic or recreational road; today, rural major access road
Design Speed:	45 mph [70 km/h]
Design Traffic Volume:	1,200 vehicles per day (seasonal)

Cross Section

The existing total roadway width within the project over most of its length is 18 ft [5.4 m]. This does not meet the roadway width guidelines of 20 ft [6.0 m] for rural scenic or recreational roads shown in Table 4-1. Furthermore, there is some evidence of sideswipe crashes, especially involving boat trailers, and one head-on collision occurred on the roadway last year. Therefore, the engineer decides that the existing roadway should be widened. The engineer decides to widen the roadway to 28 ft [8.4 m], which is appropriate for a rural major access road and will also better accommodate bicycles.

Horizontal Alignment

For improvement projects to existing roads, the guidelines suggest that, for curves with lower speeds, there is no need to change the existing curve geometry and cross section or to make other improvements if:

1. The nominal design speed of the curve is within 20 mph or 30 km/h of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific crash pattern associated with the curve.

The roadway meets these criteria, as the roadway is fairly straight with only a few horizontal curves with radii greater than 1,000 ft, and none of the reported crashes were related to a horizontal curve.

Stopping Sight Distance

There is no evidence of a site-specific crash pattern related to stopping sight distance. Therefore, no modifications will be made to the horizontal and vertical alignments.

Intersection Sight Distance

There is no evidence of a site-specific crash pattern related to intersection sight distance. Therefore, no modifications will be made to increase intersection sight distance at any of the intersections along the rehabilitation project.

Roadside Design

There is no evidence of a site-specific crash pattern indicating the desirability of providing a wider roadside clear zone or guardrail. Therefore, no improvements will be made to roadside design.

5.10 EXAMPLE 10—RESURFACING OF AN URBAN MAJOR ACCESS ROAD

A city engineer is about to begin a resurfacing project on a major access road with a design speed of 40 mph [60 km/h]. The road is functionally classified as a minor collector and provides access to adjoining property as well as to several minor access roads. The roadway currently has an average daily traffic volume of 1,500 vehicles per day and is expected to grow to 1,800 vehicles per day over the design life of the new surface. Because the roadway is classified as a minor collector road and the project is not new construction, the city engineer has concluded that it is appropriate to use the design guidelines for low-volume roads presented in Chapter 4.

The crash history of the roadway is below what is expected for similar roadways in the city, with only a few property damage crashes occurred in the past 5 years. Parking is not permitted on the roadway, which is currently 24 ft [7.2 m] wide. There are approximately three dwelling units per acre along the road.

In summary, the traffic engineer responsible for this state project knows the following information before the resurfacing process begins:

Project Type:	resurfacing of an existing road
Area Type:	suburban, medium development density
Functional Subclass:	urban major access road
Design Speed:	40 mph [60 km/h]
Design Traffic Volume:	1,800 vehicles per day

Cross Section

The existing total roadway width within the project is 24 ft [7.3 m]. While the roadway width is less than that indicated in the Table 4-1 guidelines for new construction of rural major access roads (which are also applicable to new construction of urban major access roads), the existing roadway has a curb-and-gutter section, rather than shoulders, which is acceptable for lower-speed urban streets. Furthermore, no site-specific crash patterns attributable to cross section width have been found at the site and no parking is permitted along the road; therefore, the engineer concluded that existing total roadway width of 24 ft [7.3 m] may remain in place.

Horizontal Alignment

For improvement projects to existing roads, the guidelines suggest that for curves on low-volume roads with lower speeds, reconstruction without changing the existing curve geometry and cross section or making other improvements is acceptable if:

1. The nominal design speed of the curve is within 20 mph or 30 km/h of the design or operating speed.
2. There is no clear evidence of a site-specific crash pattern associated with the curve.

The engineer finds that both of these criteria are met, so no improvements to the horizontal alignment are needed.

Stopping Sight Distance

There is no evidence of a site-specific crash pattern related to stopping sight distance. Therefore, no modifications will be made to the horizontal and vertical alignments.

Intersection Sight Distance

There is no evidence of a site-specific crash pattern related to intersection sight distance. Therefore, no modifications will be made to increase intersection sight distance at any of the intersections along the resurfacing project.

Roadside Design

There is no evidence of a site-specific crash pattern indicating the desirability of providing a wider roadside clear zone or guardrail. Therefore, no improvements will be made to the roadside design.

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American Association of State Highway and Transportation Officials
444 N. Capitol Street, NW, Suite 249 • Washington, DC 20001
202-624-5800 • Fax: 202-624-5806

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