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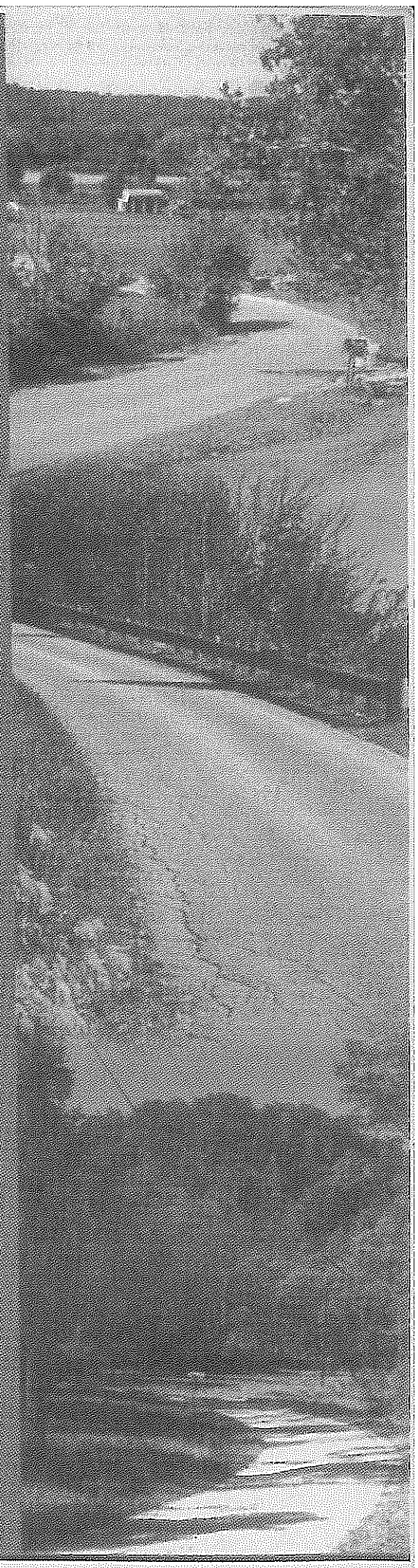

American Association of
State Highway and
Transportation Officials

Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)

2001

American Association of
State Highway and
Transportation Officials

Bradley L. Mallory, President
Secretary
Pennsylvania Department
of Transportation

John Horsley
Executive Director

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to the *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)*.

Please replace the existing text with the following revisions to ensure that your edition is both accurate and current.

AASHTO staff sincerely apologizes for any inconvenience.

AASHTO Publications Staff
August 2002

Technical Corrections

(1) On page 45, replace Exhibit 14 with the following table:

Metric		US Customary	
Design speed (km/h)	Sight distance (m)	Design speed (mph)	Sight distance (ft)
20	20	15	70
30	25	20	90
40	35	25	115
50	45	30	140
60	55	35	165
70	65	40	195
80	75	45	220
90	90	50	245
100	105	55	285
		60	325
Note: For approach grades greater than 3%, multiply the sight distance value by the appropriate adjustment factor from Exhibit 15.			

Exhibit 14. Recommended Sight Distance Guidelines for New Construction of Intersections with No Traffic Control (Case A) (1, 6)

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(2) Also, in the last line of text of the first paragraph on page 45, change "80 m [255 ft] and 40 m [120 ft]" to "75 m [245 ft] and 45 m [140 ft]" so that the text reflects the revisions that were made in Exhibit 14.

(3) On page 46, replace Exhibit 15 with the following table:

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

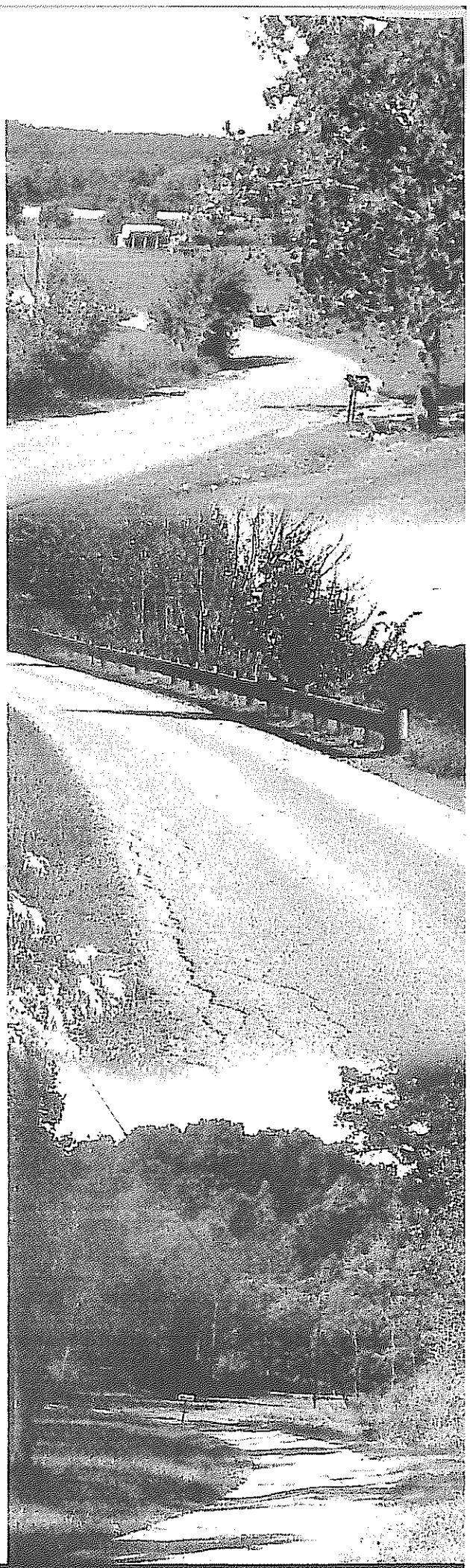
US Customary										
Approach grade (%)	Design speed (mph)									
	15	20	25	30	35	40	45	50	55	60
–6	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
–5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
–4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
–3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	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.

Exhibit 15. Adjustment Factors for Sight Distance Based on Approach Grade (1, 6)

Guidelines for

Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)



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Preface

These Guidelines were developed as part of the continuing work of the Standing Committee on Highways. The Committee, then titled the Committee on Planning and Design Policies, was established in 1937 to formulate and recommend highway engineering policies. This Committee 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, and 2001; *A Policy on Design Standards—Interstate System*, 1956, 1967, and 1991; and a number of other AASHO and AASHTO policy and "guide" publications.

An AASHTO publication is typically developed through the following steps: (1) The Committee selects subjects and broad outlines of material to be covered. (2) The appropriate subcommittee and its task forces, in this case, the Subcommittee on Design and its Task Force 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 Subcommittee, until agreement is reached. (3) The manuscript is then submitted for approval by the Standing Committee on Highways. 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.

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Chapter 1
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Chapter 2
FRAMEWORK FOR DESIGN GUIDELINES

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Foreword

As highway designers, highway 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 the design problems. The geometric design of very low-volume local 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 guidance supplied by this text, *Geometric Design Guidelines for Very Low-Volume Local Roads (ADT ≤ 400)*, 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*, also known as the Green Book. The guidance presented here will be incorporated in a future edition of that policy.

The guidelines for geometric design of very low-volume local roads are 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 FHWA. After completion of the NCHRP research, these guidelines went through the normal AASHTO review process. During the development process, 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 have participated in review of the guidelines.

Design values are presented in this document in both metric and U.S. customary units and were developed independently within each system. The relationship between the metric and U.S. customary values is neither an exact (soft) conversion nor a completely rationalized (hard) conversion. The metric values are those that would have been used had the policy been presented exclusively in metric units; the U.S. customary values are those that would have been used if the policy had been presented exclusively in U.S. customary 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* 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 very low-volume roads which experience substantially fewer crashes than higher volume roads. These guidelines recommend an approach to geometric design for very 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 and on reviews of site-specific safety conditions.

These guidelines address issues for which appropriate geometric design guidance for very low-volume local 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.*

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.

The highway, vehicle, and individual users are all integral parts of transportation safety and efficiency. While this document primarily addresses geometric design issues, a properly equipped and maintained vehicle and reasonable and prudent performance by the user are also necessary for safe and efficient operation of the transportation facility.

CHAPTER 1
INTRODUCTION

This document presents geometric design guidelines for very low-volume local roads. The purpose of the guidelines is to help highway designers in selecting appropriate geometric designs for roads with low traffic volumes traveled by motorists who are generally familiar with the roadway and its geometrics. The design guidelines presented here may be used on very low-volume local roads in lieu of the applicable policies for design of local roads and streets presented in AASHTO *Policy on Geometric Design of Highways and Streets* (**1**), commonly known as the Green Book.

This chapter defines very low-volume local 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.

DEFINITION OF VERY LOW-VOLUME LOCAL ROADS

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

> A *very low-volume local road* is a road that is functionally classified as a local road and has a design average daily traffic volume of 400 vehicles per day or less.

The preceding statement clarifies that the functional classification of a road is a key element of the definition of a very low-volume local road. 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. 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 roads may be under Federal, state, or local control.

The guidelines presented in this document may also be applied in the design of roads that are functionally classified as collectors, so long as the road has a design average daily traffic volume of 400 vehicles per day or less and primarily serves drivers who are familiar with the roadway. There are roads in some states that, because of their length and position in the road network, are functionally classified as collectors, even though they serve very low volumes of primarily local or repeat drivers. Collector roads, by their nature, serve more through traffic than local roads; however, much of that through traffic consists of familiar drivers moving between local roads and arterials. The risk assessment on which the design guidelines are based is applicable to any roadway with design average daily traffic volume of 400 vehicles per day or less that serves primarily familiar drivers. Therefore, throughout the remainder of this document, when reference

is made to very low-volume local roads, it should be understood that the guidelines are also applicable to very low-volume collector roads that primarily serve familiar drivers.

Nearly 80 percent of the roads in the United States have traffic volumes of 400 vehicles per day or less. The very low-volume local and collector roads, defined above, to which the guidelines presented in this document are applicable, should include most of this extensive road mileage. In some states, portions of the state numbered route system meet the definition of very low-volume collector roads that serve familiar drivers and can be addressed with these guidelines.

SCOPE OF GUIDELINES

The guidelines presented in this document are intended for application in the design of very low-volume local roads, as defined above, including application in both new construction and in the improvement of existing roads. The scope of the guidelines includes roads in both rural and urban areas.

The design guidelines enable designers for projects on very low-volume local roads to apply design criteria that are less restrictive than those generally used on higher volume roads. The risk assessment on which the guidelines are based shows that these less restrictive design criteria can be applied on very low-volume local roads without compromising safety. 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 safety benefits. 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 and vehicle mix on the roadway should also be considered.

The design guidelines are intended to encourage rational safety management practices on very low-volume local roads. Expenditures for safety improvements are discouraged at sites where such improvements are likely to provide little safety benefit, but are strongly encouraged at sites where safety problems 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 necessary to correct documented safety problems.

The scope of the 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.

RELATIONSHIP TO OTHER AASHTO POLICIES

The design guidelines presented in this document may be applied to very low-volume roads in lieu of the applicable policies of the AASHTO *Policy on Geometric Design of Highways and Streets* (1) and the AASHTO *Roadside Design Guide* (2). These design guidelines may be applied in place of Chapter 5 (Local Roads and Streets) of the AASHTO *Policy on Geometric Design of Highways and Streets* (1) to local roads that serve traffic volumes of 400 vehicles per day or less. For very low-volume collector roads that primarily serve familiar drivers, these design guidelines may be applied in lieu of the applicable policies in Chapter 6 (Collector Roads and Streets) of the AASHTO *Policy on Geometric Design of Highways and Streets* (1). The design guidelines presented here address design issues for which an explicit safety 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 *Policy on Geometric Design of Highways and Streets* (1) and the AASHTO *Roadside Design Guide* (2).

ORGANIZATION OF THIS DOCUMENT

Chapter 2 of this document presents a framework for design guidelines that helps to define the situations in which specific design guidelines should be applied.

Chapter 3 describes the design philosophy on which the guidelines are based. Specifically, this chapter identifies the unique characteristics of very low-volume local roads, the basis for the design recommendations, the risk assessment approach used in their development, the applicability of the guidelines to new construction and to improvement projects on existing roads, and the flexibility provided to designers by these guidelines.

Chapter 4 presents the design guidelines applicable to cross section elements, bridge width, horizontal alignment, stopping sight distance, intersection sight distance, roadside design, unpaved roads, and two-way single-lane roads.

Chapter 5 presents examples of the application of the design guidelines to specific design situations.

CHAPTER 2
FRAMEWORK FOR DESIGN GUIDELINES

This chapter presents a framework for the design guidelines for very low-volume local roads. The elements of this framework are area type, roadway functional class and subclass, design speed/operating speed, and traffic volume. The chapter identifies how these elements of the framework are used in identifying the appropriate design guidelines for a specific design application.

AREA TYPE

The design guidelines are applicable to both rural and urban areas. The operating characteristics, constraints, and configurations of very 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.

Very low-volume local 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.

By contrast, urban and suburban roads, even those with very 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.

FUNCTIONAL CLASSIFICATION

The concept of functional classification is fundamental to the criteria used in 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 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, and 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 roads with traffic volumes of 400 vehicles per day or less. For purposes of the design guidelines, these very low-volume local 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/commercial access roads
- rural agricultural access roads
- rural recreational and scenic roads
- rural resource recovery roads

Urban Roads

- urban major access streets
- urban residential streets
- urban industrial/commercial access streets

Each of these functional subclasses is defined below.

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 may thus, in some respects, function like collector or even minor arterial roads, particularly since even arterials often carry low traffic volumes in rural areas. Major access roads are usually paved, but may be unpaved in some rural areas. As discussed in Chapter 1, the design guidelines for very low-volume local roads may also be applied to some collector roads that primarily serve familiar drivers. Such collector roads should be treated as major access roads for purposes of these guidelines.

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 classified separately.

Rural Industrial/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/commercial access roads are very short, and in many cases they do not serve any through traffic. Industrial/commercial access roads may be either paved or unpaved. Such roads are classified separately from minor access roads, which they otherwise resemble, because consideration of trucks and other heavy vehicles is important in their design.

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 2.6-m [8.5-ft] truck. Roads that provide frequent access to farms for conventional trucks, but not for wider equipment, should be treated as rural commercial/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.

Rural Recreational and Scenic Roads

Recreational and scenic roads serve specialized land uses, including parks, tourist attractions, and recreation facilities, such as campsite 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 in speeds and trip lengths may be fairly long. Such roads can be either paved or unpaved.

Rural Resource Recovery Roads

Resource recovery roads are local roads serving logging or mining operations. Such roads are typically found only in rural areas. Resource recovery roads are distinctly different from the other functional subclasses of very low-volume local 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.

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 slightly more through traffic than most local roads is much the same. Thus, urban major access streets approach the status of a minor collector. As discussed in Chapter 1, the design guidelines for very low-volume local streets may also be applied some collector streets that primarily serve familiar drivers. Such collector streets should be treated as major access streets for purposes of the guidelines.

Urban Residential Streets

Urban residential streets typically serve to provide access to single- and multiple-family residences in urban areas. Motorists 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 fire trucks and school buses is an important consideration in the design of residential streets.

Urban Industrial/Commercial Access Streets

Urban industrial/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/commercial site to the local or regional highway network. Industrial/commercial 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/commercial street is that its design is influenced by the heavy vehicles using the street.

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.

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.

DESIGN SPEED/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 10 km/h [5 mph]. Designers select a design speed which is appropriate for the roadway and is used to correlate the various features of the design. The selected design speed should realistically represent actual or anticipated operating speeds and conditions on the roadway being designed.

Several of the design guidelines presented in Chapter 4 differ as a function of speed, as follows:

- Low speed—0 to 70 km/h [0 to 45 mph]
- High speed—more than 70 km/h [45 mph]

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 about 20 years from the date of completion of construction, but may range from the current year to 20 years 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 very low-volume local 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 traffic volume above 400 vehicles per day is anticipated on a local road within the period for which traffic volumes are projected, then Chapter 5 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1) should be used instead of the design guidelines presented here. Where future development is uncertain, a project with a projected volume of 400 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.

Traffic volumes on very low-volume roads are stratified into three levels for purposes of the design guidelines in Chapter 4. The volume ranges are:

- 100 vehicles per day or less
- 100 to 250 vehicles per day
- 250 to 400 vehicles per day

CHAPTER 3
DESIGN PHILOSOPHY

This chapter presents the design philosophy on which the design guidelines in Chapter 4 are based. The presentation of the design philosophy includes a discussion of the unique characteristics of very low-volume local roads, a discussion of the basis for the design guidelines, the risk assessment approach used to develop the guidelines, the differences between the guidelines for new construction and for improvement of existing roads, and the need for flexibility in applying the design guidelines.

UNIQUE CHARACTERISTICS OF VERY LOW-VOLUME LOCAL ROADS

The design guidelines presented in Chapter 4 are based on the unique characteristics of very low-volume local roads. The fundamental characteristics of very low-volume local roads that distinguish them from other types of roads are:

- The traffic volumes of such roads are, by definition, very low. All very low-volume local roads have average daily traffic volumes of 400 vehicles per day or less, and many such roads have volumes that are much less than the 400-vehicle-per-day threshold value. These very 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 local nature of the road means that most 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 very low-volume local roads can be less stringent than those used for higher volume roads or roads that serve primarily unfamiliar drivers. The functional subclasses of very low-volume local roads presented in Chapter 2 permit the design guidelines to vary with the expected proportion of unfamiliar drivers. Similarly, design guidelines for very low-volume local roads also vary with the expected design traffic volume level.

BASIS FOR DESIGN RECOMMENDATIONS

Design criteria for streets, roads, and highways are based on a wide range of considerations. Operational quality, safety, constructability, and maintainability are of primary importance. While safety 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 highway should reflect the potential for changes in traffic volumes, patterns, and operating conditions. Similarly, a wide

range of driver and vehicle characteristics may use a highway, including unfamiliar or less skilled drivers, and a combination of passenger cars, trucks, and other vehicle types.

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 very 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 are developed to be "safety conservative." In other words, design for basic geometric elements such as alignment and cross section have been historically derived to provide a "margin of safety" to reflect 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 very low-volume local roads presented in this document are based on a safety risk assessment performed by Neuman (**3**). This risk assessment was intended to establish design criteria for very low-volume local roads that, when applied systemwide, will have margins of safety that are comparable to those presented in the AASHTO *Policy on Geometric Design of Highways and Streets* (**1**) for higher volume roads. However, because of the unique characteristics of very low-volume local 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 (**1, 2**), TRB Special Report 214 (**4**), NCHRP Report 362 (**5**), NCHRP Report 383 (**6**), NCHRP Report 400 (**7**), horizontal curve research by Zegeer et al. (**8**), guardrail research by Stephens (**9**) and by Wolford and Sicking (**10**), design guidelines developed by the United States Forest Service (**11**) and the Transportation Association of Canada (**12**), and the *Recommended Guidelines for Subdivision Streets* (**13**) developed by the Institute of Transportation Engineers (ITE).

An important component of the design guidelines for very low-volume local 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.

DEVELOPMENT OF DESIGN GUIDELINES THROUGH RISK ASSESSMENT

The risk assessment by Neuman (3) recommends that design criteria for very low-volume local roads should be based on tradeoffs between two factors:

- demonstrable differences in construction and maintenance costs
- estimated impacts on traffic crash frequency or severity

This approach highlights safety 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 very low-volume local roads to influence their fundamental design criteria.

Because it is derived from a formal risk assessment, the design philosophy recommended for very low-volume local roads is based fundamentally on safety concerns. Moreover, the philosophy focuses on direct comparison of known or expected safety benefits and system costs. This tradeoff implies that public funds spent to improve such roads in the name of safety should be spent only where there is likely to be an actual safety benefit in return. This, in turn, assures that highway funds expended for safety purposes on all highways (not just low-volume local roads) will be available for use where they are most needed (i.e., where meaningful safety benefits can reasonably be expected).

Risk Assessment Approach

The risk assessment represents a comparison between crash risk for very low-volume local 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 *Policy on Geometric Design of Highways and Streets* (1). The guidelines concerning threshold or acceptable risk levels for new construction of very low-volume local roads used by Neuman (3) were:

- For urban or low-speed facilities, an acceptable safety 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 safety 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.

These risk assessment thresholds for rural and urban roadways are consistent with those used to evaluate roadway widths in NCHRP Report 362 (5), which was the basis for the current lane and shoulder width design values for rural highways in the AASHTO *Policy on Geometric Design of Highways and Streets* (1). Although NCHRP Report 362 considers roadways with

higher ADTs than those addressed in these guidelines, it provided a model for the risk assessment of very low-volume local roads.

The acceptable risk levels represented by the thresholds presented above were applied by Neuman (3) 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, use in a project of a minimum radius curve 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 *Policy on Geometric Design of Highways and Streets* (1), but the safety performance 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 100 km/h [60 mph] 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 *Policy on Geometric Design of Highways and Streets* (1), is 435 m [1340 ft]. Neuman (3) determined that the difference in safety performance between a curve with a radius of 435 m [1340 ft] and a curve with a radius of 250 m [830 ft] for the specified conditions would be less than one crash per km [1.6 crashes per mile] over a period of 10 years. Therefore, the use of a minimum radius of 250 m [830 ft] can be recommended for horizontal curves on very low-volume local 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 (3).

This example does not imply that the minimum radius of 250 m [830 ft] is appropriate for all horizontal curves on very low-volume local roads, any more than the minimum radius of 435 m [1340 ft] 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 250 m [830 ft] 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 improvement in systemwide safety. The safety performance of most of the existing very low-volume local road system will be unaffected by the guidelines, but improvements that should have a positive effect on safety are recommended at locations on existing roads where site-specific safety problems are found.

Expected Systemwide Safety Effects

Overall, the net effect on systemwide safety of applying these design guidelines for very low-volume local roads will undoubtedly be very small. Crash rates on newly constructed roads might be slightly higher for roads designed in accordance with these guidelines than for roads designed in accordance with Green Book Chapter 5. 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 improvement in safety because existing roads with site-specific safety problems will be improved. Thus, the net effect of applying these design guidelines systemwide for very low-volume local roads should be a change in safety performance 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 very low-volume local roads on safety improvements at locations where it can be expected to provide substantial safety benefits and to discourage spending at locations where little or no safety 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.

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 very low-volume local roads. These design criteria are generally less restrictive than those used in new construction of higher volume roads such as those of the AASHTO *Policy on Geometric Design of Highways and Streets* (1).

Projects on existing very low-volume local 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 safety problem that can potentially be corrected by a roadway or roadside improvement. Where documentable site-specific safety problems do not exist, it is unlikely that any roadway or roadside improvement would provide substantial safety 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. Design decisions concerning projects on existing roads that might, on higher volume roads, be based on the design criteria in TRB Special Report 214 (4) may, on very low-volume local roads, be based on the guidelines in Chapter 4.

There are a wide variety of sources that may be considered in investigating and documenting the existence of site-specific safety problems. These naturally include crash history data. Because

very low-volume local roads have very 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 safety problems 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 safety problems because, as stated above, safety assessments of very low-volume local 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.

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 compromising safety, or to increase the design criteria to the levels used for higher volume roads in the AASHTO *Policy on Geometric Design of Highways and Streets* (1).

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 safety problems and to focus safety expenditures on those sites where a site-specific safety problem 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 safety problem 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 required; 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, and 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.

CHAPTER 4
DESIGN GUIDELINES

This chapter presents design guidelines for specific aspects of the design of very low-volume local roads including cross section (traveled way and shoulder widths), horizontal alignment, stopping sight distance, intersection sight distance, roadside design, unpaved roads, and two-way single-lane roads.

CROSS SECTION

The key elements of cross section design for a roadway are traveled way width and 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 section in new construction projects and on existing very low-volume local roads are presented below.

New Construction

The design guidelines for cross section in new construction projects on very low-volume local 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.

Very Low-Volume Local Roads in Rural Areas

Exhibit 1 presents the guidelines for total roadway widths for newly constructed roads in rural areas. The total roadway width criteria vary from 5.4 to 8.0 m [18 to 26 ft] with the functional subclass and the design speed of the road. These values were developed in research by Neuman (**3**) from several sources. The primary source for cross section widths was NCHRP Report 362 (**5**); other sources included TRB Special Report 214 (**4**), the United States Forest Service (USFS) (**11**), and the Transportation Association of Canada (**12**).

Metric						
	Total roadway width (m) by functional subclass					
Design speed (km/h)	Major access	Minor access	Recreational and scenic	Industrial/ commercial access	Resource recovery	Agricultural access
20	–	5.4	5.4	6.0	6.0	6.6
30	–	5.4	5.4	6.0	6.0	7.2
40	5.4	5.4	5.4	6.4	6.4	7.2
50	5.4	5.4	5.4	6.8	6.8	7.2
60	5.4	5.4	5.4	6.8	6.8	7.2
70	6.0	6.0	6.0	7.0	–	8.0
80	6.0	6.0	6.0	7.4	–	–
90	6.6	–	6.6	–	–	–
100	6.6	–	–	–	–	–
US Customary						
	Total roadway width (ft) by functional subclass					
Design speed (mph)	Major access	Minor access	Recreational and scenic	Industrial/ commercial access	Resource recovery	Agricultural access
15	–	18.0	18.0	20.0	20.0	22.0
20	–	18.0	18.0	20.0	20.0	24.0
25	18.0	18.0	18.0	21.0	21.0	24.0
30	18.0	18.0	18.0	22.5	22.5	24.0
35	18.0	18.0	18.0	22.5	22.5	24.0
40	18.0	18.0	20.0	22.5	–	24.0
45	20.0	20.0	20.0	23.0	–	26.0
50	20.0	20.0	20.0	24.5	–	–
55	22.0	–	22.0	–	–	–
60	22.0	–	–	–	–	–
Note: Total roadway width includes the width of both traveled way and shoulders.						

Exhibit 1. Guidelines for Total Roadway Width for New Construction of Very Low-Volume Local Roads in Rural Areas

The cross section width guidelines for major access roads, minor access roads, and recreation and scenic roads are based primarily on travel by passenger cars and recreational vehicles. Widths for industrial/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/commercial access roads, resource recovery roads, and agricultural access roads reflect the offtracking and maneuverability requirements 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/commercial access roads, resource recovery roads, and agricultural access roads should not be construed as a safety requirement. It should be noted that the roadway widths for agricultural access roads are applicable on roads used by agricultural equipment wider than a typical 2.6-m [8.5-ft] truck.

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 Exhibit 1 may be appropriate adjacent to historic structures or in mountainous terrain. In determining appropriate roadway widths, the designer should refer to the discussion of design flexibility in Chapter 3.

Designers should be afforded great discretion in the use of Exhibit 1, even for new construction. Small differences in the existing or proposed dimensions from those shown in Exhibit 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 Exhibit 1 accordingly.

Very Low-Volume Local Roads in Urban Areas

As in rural areas, the cross section width guidelines for very low-volume local roads in urban areas are related to basic operational requirements. Speeds are lower, trip lengths and lengths of local roads are generally much shorter, and available right-of-way width is much less than in rural areas. The major functional requirements for very low-volume local 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/commercial access roads should generally be the same as those shown for comparable rural roads in Exhibit 1. Greater widths are desirable where parking is permitted.

Cross section width guidelines for urban residential streets are shown in Exhibit 2. These widths incorporate consideration of access for fire trucks and other emergency vehicles and apply to streets where parking is permitted on both sides of the roadway. Reduced widths may be appropriate where parking is restricted. These guidelines are based on the ITE *Recommended Guidelines for Subdivision Streets* (**13**).

Development density	Metric Total roadway width (m)	US Customary Total roadway width (ft)
Low	6.1 to 8.5	20 to 28
Medium	8.5 to 10.3	28 to 34
Note: Low development density represents 2.0 or fewer dwelling units per acre; medium development density represents 2.1 to 6.0 dwelling units per acre.		

Exhibit 2. Guidelines for Total Roadway Width for New Construction of Urban Residential Streets

The lower end of the range of residential street widths in the ITE guidelines presented in Exhibit 2 are applicable to subdivision streets with sufficient off-street parking (e.g., driveways and garages) so that on-street parking is used only occasionally by visitors and delivery vehicles. The higher end of the range of street widths is applicable where there is frequent parking on one side of the street. On streets with frequent parking on both sides of the street, street widths greater than those shown in Exhibit 2 may be appropriate.

Design criteria for curbs and sidewalks on very low-volume urban roads and streets should be determined based on local policies and published guidelines for compliance with the Americans with Disabilities Act (ADA).

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 safety problem. Chapter 3 discusses the types of evidence of a site-specific safety problem that might be considered. When a site-specific safety problem that can be mitigated by a wider roadway is identified, the cross section for the portion of the roadway with the identified safety problem should be widened to at least the total roadway widths presented above for new construction.

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 safety performance of the existing bridge. Determination of bridge widths for newly constructed bridges and existing bridges is addressed below.

New Construction

Newly constructed bridges are bridges on new roadways where there is no existing roadway or bridge in place. The widths of newly constructed bridges should generally be selected in

accordance with the bridge width criteria for local roads in Chapter 5 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1). Those criteria state that, for bridges on local roads with ADT of 400 veh/day or less, the bridge width should be equal to the width of the traveled way plus 0.6 m [2 ft]. 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 30 m [100 ft] in length should be evaluated individually to determine the appropriate bridge width. Bridge usage by trucks and recreational vehicles 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 ADT less than 100 veh/day where the designer finds that a one-lane bridge can operate effectively. The minimum width of a one-lane bridge should be 4.5 m [15 ft] unless the designer concludes that a narrower bridge can function effectively (e.g., based on the safety performance of similar bridges maintained by the same agency). Caution should be exercised in design of one-lane bridges wider than 4.9 m [16 ft] 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 intervisible pull-offs at each end where drivers can wait for traffic on the bridge to clear.

Existing Bridges

Existing bridges can remain in place without widening unless there is evidence of a site-specific safety problem related to the width of the bridge. As described in Chapter 3, evidence of a site-specific safety problem 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 safety problem, 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.

HORIZONTAL ALIGNMENT

For balance in roadway design, all geometric elements should, as far as economically practical, be designed to provide safe, 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. In the design of roadway curves, it is necessary to establish proper 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:

Metric	US Customary
$f = \frac{V^2}{127R} - 0.01e$	$f = \frac{V^2}{15R} - 0.01e$ **(1)**
where: f = side friction factor V = vehicle speed (km/h) R = radius of curve (m) e = rate of roadway superelevation (percent)	where: f = side friction factor V = vehicle speed (mph) R = radius of curve (ft) 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 (1) can be recast as:

Metric	US Customary
$R_{min} = \frac{V^2}{127(0.01e_{max} + f_{max})}$	$R_{min} = \frac{V^2}{15(0.01e_{max} + f_{max})}$ **(2)**
where: R_{min} = minimum curve radius (m) e_{max} = maximum rate of superelevation permitted by highway agency policy f_{max} = maximum side friction factor	where: R_{min} = minimum curve radius (ft) e_{max} = maximum rate of superelevation permitted by highway agency policy f_{max} = maximum side friction factor

The values of f_{max} and R_{min} used in design of most higher volume roads are specified in Chapter 3 of the AASHTO *Policy on Geometric Design of Highways and Streets* (**1**) and are presented here in Exhibit 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 *Policy on Geometric Design of Highways and Streets* (**1**). The values of f_{max} in Exhibit 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 of safety against loss of control due to skidding on most pavements, even at high speeds.

Metric						
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
20	0.180	15	15	10	10	10
30	0.170	35	30	30	25	25
40	0.170	60	55	50	45	45
50	0.160	100	90	80	75	70
60	0.150	150	135	125	115	105
70	0.140	215	195	175	160	150
80	0.140	280	250	230	210	195
90	0.130	375	335	305	275	255
100	0.120	490	435	395	360	330
US Customary						
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
15	0.175	70	65	60	55	50
20	0.170	125	115	105	100	90
25	0.165	205	185	170	160	145
30	0.160	300	275	250	230	215
35	0.155	420	380	350	320	300
40	0.150	565	510	465	430	395
45	0.145	730	660	600	555	510
50	0.140	930	835	760	695	645
55	0.130	1190	1065	965	880	810
60	0.120	1505	1340	1205	1095	1005

Exhibit 3. Maximum Side Friction Factor and Minimum Radius for Horizontal Curve Design on Higher Volume Roadways (1)

Exhibit 4 presents the values of f_{max} and R_{min} used in design of higher volume low-speed urban streets, as specified in Chapter 3 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1). These criteria are applicable to urban streets with design speeds of 70 km/h [45 mph] or less. Superelevation rates greater than 6 percent are not recommended for such streets because higher rates would be inappropriate for low-speed operation.

Metric			
Design speed (km/h)	Maximum design side friction factor, f_{max}	Minimum radius (m), R_{min} Max. superelevation rate (%), e_{max}	
		4	6
20	0.350	10	10
30	0.312	20	20
40	0.252	45	40
50	0.214	80	70
60	0.186	125	115
70	0.163	190	175
US Customary			
Design speed (mph)	Maximum design side friction factor, f_{max}	Minimum radius (ft), R_{min} Max. superelevation rate (%), e_{max}	
		4	6
15	0.330	40	40
20	0.300	80	75
25	0.252	145	135
30	0.221	230	215
35	0.197	345	320
40	0.178	490	450
45	0.163	665	605

Exhibit 4. Maximum Side Friction Factor and Minimum Radius for Horizontal Curves on Higher Volume Low-Speed Urban Streets (1)

A risk assessment by Neuman (3) 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 very low-volume local roads with no discernable degradation in safety. The specific criteria applicable to horizontal curve design for new construction projects and for existing very low-volume local roads are presented below.

New Construction

The following guidelines are recommended for design of horizontal curves in new construction of very low-volume local roads:

- For the design of very low-volume local roads without substantial truck recreational and vehicle volumes, acceptable operations can be obtained with smaller curve radii than those shown in Exhibit 3. Design radii based on a reduction in design speed of 10 to 20 km/h, or 5 to 10 mph, may be used. The maximum reduction in design speed of 20 km//h or 10 mph is generally appropriate for roadways with speeds of 70 km/h [45 mph] or more and with average daily traffic volumes of 250 vehicles per day or

less. For roadways with average daily traffic volumes of 250 to 400 vehicles per day without substantial truck volumes, the appropriate maximum reduction in design speed is 15 km/h or 10 mph.

- For the design of very low-volume local roads carrying substantial recreational vehicle and truck traffic, design radii based on no reduction in design speed should be used at very low speeds (e.g., 20 km/h or 15 mph). 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 10 km/h or 5 mph, may be used.

The specific guidelines for the design of horizontal curves are presented separately for six categories of very low-volume local roads. These are:

- rural major access, minor access, and recreational and scenic roads with average daily traffic volumes of 250 vehicles per day or less
- rural major access, minor access, and recreational and scenic roads with average daily traffic volumes from 250 to 400 vehicles per day
- rural industrial/commercial access, agricultural access, and resource recovery roads
- urban major access streets with average daily traffic volumes of 250 vehicles per day or less and urban residential streets
- urban major access streets with average daily traffic volumes from 250 to 400 vehicles per day
- urban industrial/commercial access streets.

Horizontal curve design criteria for new construction of roads in each of these six categories are presented below.

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 on local roads 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 Exhibit 3, whenever practical. In constrained situations, for roads with average daily traffic 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 Exhibit 5. This exhibit incorporates reductions in design speed up to 20 km/h or 10 mph based on the design principles presented above. Exhibit 5 is appropriate in constrained situations, where providing a horizontal curve designed in accordance with Exhibit 3 would require 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 very low-volume local roads is discussed later in this chapter.

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
20	20	0.180	15	15	10	10	10
30	25	0.170	25	20	20	20	20
40	30	0.170	35	30	30	25	25
50	35	0.170	45	40	40	35	35
60	45	0.165	80	70	65	60	55
70	50	0.160	100	90	80	75	70
80	60	0.150	150	135	125	115	105
90	70	0.140	215	195	175	160	150
100	80	0.140	280	250	230	210	195
US 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
15	15	0.175	70	65	60	55	50
20	15	0.175	70	65	60	55	50
25	20	0.170	125	115	105	100	90
30	20	0.170	125	115	105	100	90
35	25	0.165	205	185	170	155	145
40	30	0.160	300	275	250	230	215
45	35	0.155	420	380	350	320	295
50	40	0.150	560	510	465	425	395
55	45	0.140	750	675	615	565	515
60	50	0.140	925	835	760	695	640

Exhibit 5. Guidelines for Maximum Side Friction Factor and Minimum Radius (New Construction, ADT < 250 veh/day, Limited Heavy Vehicle Traffic)

Rural Major Access, Minor Access, and Recreational and Scenic Roads (250 to 400 Vehicles per Day)

As in the previous category, rural major access, minor access, and recreational and scenic roads with average daily traffic volumes from 250 to 400 vehicles per day should be designed with horizontal curves based on the limiting values of f_{max} and R_{min} shown in Exhibit 3, whenever practical. In constrained situations, the limiting values of f_{max} and R_{min} shown in Exhibit 6 may be used. Exhibit 6 incorporates reductions in design speed up to 15 km/h or 10 mph based on the design principles presented above. Design of superelevation and superelevation transitions for this category of very low-volume local roads is discussed later in this chapter.

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
20	20	0.180	15	15	10	10	10
30	25	0.170	25	20	20	20	20
40	30	0.170	35	30	30	25	25
50	40	0.170	60	55	50	45	45
60	50	0.160	100	90	80	75	70
70	60	0.150	150	135	125	115	105
80	65	0.145	180	160	150	135	125
90	75	0.140	245	220	200	185	170
100	85	0.135	325	290	265	240	225
US 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
15	15	0.175	70	65	60	55	50
20	15	0.175	70	65	60	55	50
25	20	0.170	125	115	105	100	90
30	20	0.170	125	115	105	100	90
35	30	0.160	300	275	250	230	215
40	35	0.155	420	380	350	320	295
45	40	0.150	560	510	465	425	395
50	45	0.140	750	675	615	565	520
55	50	0.140	925	835	760	695	640
60	55	0.130	1190	1060	960	875	805

Exhibit 6. Guidelines for Maximum Side Friction Factor and Minimum Radius (New Construction, ADT from 250 to 400 veh/day, Limited Heavy Vehicle Traffic)

Rural Industrial/Commercial Access, Agricultural Access, and Resource Recovery Roads

Horizontal curves on rural industrial/commercial access, agricultural access, and resource recovery roads should be designed using the limiting values of f_{max} and R_{min} shown in Exhibit 3, whenever practical. In constrained situations, the limiting values of f_{max} and R_{min} shown in Exhibit 7 may be used. Exhibit 7 incorporates reductions in design speed up to 10 km/h or 5 mph. Lower

Metric							
Design speed (km/h)	Reduced design speed (km/h)	Maximum side friction factor, f_{max}	Minimum radius (m), R_{min} Max. superelevation rate (%), e_{max}				
			4	6	8	10	12
20	20	0.180	15	15	10	10	10
30	30	0.170	35	30	30	25	25
40	40	0.170	60	55	50	45	45
50	45	0.165	80	70	65	60	55
60	55	0.155	120	110	100	95	85
70	65	0.145	180	160	150	135	125
80	70	0.140	215	195	175	160	150
90	80	0.140	280	250	230	210	195
100	90	0.130	375	335	305	275	255
US Customary							
Design speed (mph)	Reduced design speed (mph)	Maximum side friction factor, f_{max}	Minimum radius (ft), R_{min} Max. superelevation rate (%), e_{max}				
			4	6	8	10	12
15	15	0.175	70	65	60	55	50
20	20	0.170	125	115	105	100	90
25	25	0.165	205	185	170	155	145
30	25	0.165	205	185	170	155	145
35	30	0.160	300	275	250	230	215
40	35	0.155	420	380	350	320	295
45	40	0.150	560	510	465	425	395
50	45	0.140	750	675	615	565	520
55	50	0.140	925	835	760	695	640
60	55	0.130	1190	1060	960	895	805

Exhibit 7. Guidelines for Maximum Side Friction Factor and Minimum Radius (New Construction, ADT ≤ 400 veh/day, Substantial Proportions of Heavy Vehicle Traffic)

reductions in design speed are used for industrial/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 very low-volume local roads is discussed later in this chapter.

Urban Major Access Streets (250 Vehicles per Day or Less) and Urban Residential Streets

Horizontal curves on urban major access streets with average daily traffic volumes of 250 vehicles per day or less and on 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 Exhibit 3, whenever practical. On streets with design speeds of 70 km/h [45 mph] or less, the design criteria in Exhibit 4 may be used in preference to Exhibit 3. In constrained situations, the limiting values of f_{max} and R_{min} shown in Exhibit 5 may be used in place of Exhibits 3 and 4. Design of superelevation and superelevation transitions for this category of very low-volume local roads is discussed later in this chapter.

Urban Major Access Streets (250 to 400 Vehicles per Day)

Horizontal curves on urban major access streets with average daily traffic volumes from 250 to 400 vehicles per day should be designed in accordance with the limiting values of f_{max} and R_{min} presented in Exhibit 3, whenever practical. On streets with design speeds of 70 km/h [45 mph] or less, the design criteria in Exhibit 4 may be used in preference to Exhibit 3. In constrained situations, the limiting values of f_{max} and R_{min} shown in Exhibit 6 may be used in place of Exhibits 3 and 4. Design of superelevation and superelevation transitions for this category of very low-volume local roads is discussed later in this chapter.

Urban Industrial/Commercial Access Streets

Horizontal curves on urban industrial/commercial access streets should be designed in accordance with the limiting values of f_{max} and R_{min} presented in Exhibit 3, whenever practical. On streets with design speeds of 70 km/h [45 mph] or less, the design criteria in Exhibit 4 may be used in preference to Exhibit 3. In constrained situations, the limiting values of f_{max} and R_{min} shown in Exhibit 7 may be used in place of Exhibits 3 and 4. Design of superelevation and superelevation transitions for this category of very low-volume local roads is discussed later in this chapter.

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 *Policy on Geometric Design of Highway and Streets* (1). Where the horizontal curve design is based on Exhibits 3 or 4, 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 Exhibits 5, 6, or 7, the superelevation and superelevation transition design follow the criteria from Chapter 3 of the AASHTO Green Book using the reduced design speed indicated in Exhibits 5, 6, or 7 in place of the roadway design speed. The criteria in Chapter 3 of the AASHTO Green Book concerning situations where no superelevation is required apply to very low-volume local roads based on the roadway design speed or the reduced design speed, as appropriate.

Existing Roads

For improvement projects on existing very low-volume local roads, the existing horizontal curve geometry should generally be considered acceptable unless there is evidence of a site-specific safety problem related to horizontal curvature. The following guidelines reflect the results of the risk assessment for horizontal curves on existing roads:

- For curves on very low-volume local roads with low speeds (design or estimated operating speed of 70 km/h [45 mph] or less), reconstruction without changing the existing curve geometry and cross section is acceptable if the nominal design speed of the curve is within 30 km/h or 20 mph of the design or operating speed, and if there is no clear evidence of a site-specific safety problem associated with the curve.
- For curves on very low-volume local roads with higher speeds (design or estimated operating speed greater than 70 km/h [45 mph]), reconstruction without changing the existing curve geometry and cross section is acceptable if the nominal design speed of the curve is within 20 km/h or 10 mph of the design or operating speed, and if there is no clear evidence of a site-specific safety problem associated with the curve.

Evidence of a site-specific safety problem may be: a pattern of curve-related crashes (requiring at least 5 years, and preferably 10 years, of crash history); 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 and/or local police; or measured or known speeds substantially higher (e.g., 30 km/h or 20 mph 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 very low-volume local roads than curve flattening and reconstruction. Design actions to correct such problems should emphasize such low-cost measures and should not emphasize or encourage more costly measures such as curve flattening.

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.

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. On higher volume highways, sight distance at every point on the highway 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 extremely low traffic volumes, 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 required 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 *Policy on Geometric Design for Highways and Streets* (**1**) are based on the following model:

Metric	US Customary
$d = 0.278Vt + 0.039\frac{V^2}{a}$	$d = 1.47Vt + 1.075\frac{V^2}{a}$ **(3)**

where:
 	
d =sight distance, m;
t =brake reaction time, s;
V =design speed, km/h; and
a =driver deceleration, m/s^2
	where:
	
	d =sight distance, ft;
	t =brake reaction time, s;
	V =design speed, mph; and
	a =driver deceleration, ft/s^2

The brake reaction time (t) of 2.5 s used in Equation (3) represents approximately the 95th percentile of the observed distribution of brake-reaction times. The deceleration rate, *a*, of 3.4 m/s^2 [11.2 ft/s^2] used in Equation (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 *Policy on Geometric Design of Highways and Streets* (**1**) uses values for height of eye (h_1) and height of object (h_2) equal to 1,080 mm and 600 mm [3.5 ft and 2.0 ft], respectively.

Sight distance criteria applicable to new construction projects and to existing very low-volume local roads are presented below. The design criteria for stopping sight distance on very low-volume local 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 very low-volume local roads.

New Construction

Design of newly constructed very low-volume local roads may be based on sight distances lower than those presented in the AASHTO *Policy on Geometric Design of Highways and Streets* (1). Very low-volume local roads may be operated safely with lower sight distances because vehicles stopped in the roadway, the primary cause of crashes associated with limited sight distance on higher volume roads, are extremely rare at low volumes and because drivers familiar with a roadway are likely to anticipate locations where vehicles might be stopped or objects in the roadway might be present.

Recent research in 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 (3) concluded that vehicles stopped in the roadway were far less likely on very low-volume local roads than even the limited likelihood of such collisions on higher volume roads and that sight distance values lower than those presented in the AASHTO *Policy on Geometric Design for Highways and Streets* (1) for higher volume roads can be applied to very low-volume roads with minimal effect on safety. Based on the formal risk assessment by Neuman, two sets of alternative sight distance criteria for very low-volume local roads 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
- roads with traffic volumes of 100 to 250 vehicles per day located at lower risk locations, including locations away from intersections, narrow bridges, railroad-highway grade crossings, sharp curves, and steep downgrades

The sight distance model presented in Equation (3) using alternative parameter values for brake-reaction time and driver deceleration is recommended for the following types of very low-volume local roads:

- roads with traffic volumes of 100 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.
- roads with traffic volumes of 250 to 400 vehicles per day

The alternative parameter values recommended for use when Equation (3) is applied to very low-volume local roads are:

- a brake-reaction time of 2 s, based on the 90th rather than the 95th percentile of observed driver behavior
- a driver deceleration of 4.1 m/s^2 [13.4 ft/s^2], based on the 50th percentile rather than the 10th percentile of the observed distribution

Exhibit 8 presents recommended design sight distance criteria for very low-volume local roads based on the models discussed above. These criteria may be used in design of both horizontal and crest vertical curves for new construction of very low-volume local roads.

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, buildings, or longitudinal barriers) on the inside of a horizontal curve, a design to provide adequate sight distance may require 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, specific study is usually necessary for each condition. With the sight distance specified in Exhibit 8 for the appropriate design speed as a control, the designer should check the actual condition and make any necessary 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 at any point on the curve is the middle ordinate shown in Exhibit 9. The middle ordinate for a horizontal curve can be computed, for any curve whose length exceeds the design sight distance, as shown in Equation (4):

Metric	US Customary
$M = R\left(1 - \cos\frac{28.65\ S}{R}\right)$	$M = R\left(1 - \cos\frac{28.65\ S}{R}\right)$ (4)
where: M =middle ordinate (m) R =radius (m) S =sight distance (m)	where: M =middle ordinate (ft) R =radius (ft) S =sight distance (ft)

Exhibit 10 presents the middle ordinate that defines the width that should be clear of sight obstructions for a horizontal curve as a function of curve radius and design speed.

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 Exhibit 11. Vertical curves should be simple in application and should result in a design that is safe, comfortable in operation, pleasing

in appearance, and adequate for drainage. 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.

Metric				
	Minimum sight distance (m) for specified design traffic volumes and location types			
	0–100 veh/day	100–250 veh/day		250-400 veh/day
Design speed (km/h)	All locations	"Lower risk" locations[1]	"Higher risk" locations[2]	All locations
20	15	15	15	15
30	25	25	30	30
40	35	35	40	40
50	45	45	55	55
60	60	60	70	70
70	75	75	90	90
80	95	95	110	110
90	120	120	130	130
100	140	140	155	155
US Customary				
	Minimum sight distance (ft) for specified design traffic volumes and location types			
	0–100 veh/day	100–250 veh/day		250-400 veh/day
Design speed (mph)	All locations	"Lower risk" locations[1]	"Higher risk" locations[2]	All locations
15	65	65	65	65
20	90	90	95	95
25	115	115	125	125
30	135	135	165	165
35	170	170	205	205
40	215	215	250	250
45	260	260	300	300
50	310	310	350	350
55	365	365	405	405
60	435	435	470	470

[1] away from intersections, narrow bridges, railroad-highway grade crossings, sharp curves, and steep downgrades

[2] near intersections, narrow bridges, or railroad-highway grade crossings, or in advance of sharp curves or steep downgrades

Exhibit 8. Design Sight Distance Guidelines for New Construction of Very Low-Volume Local Roads

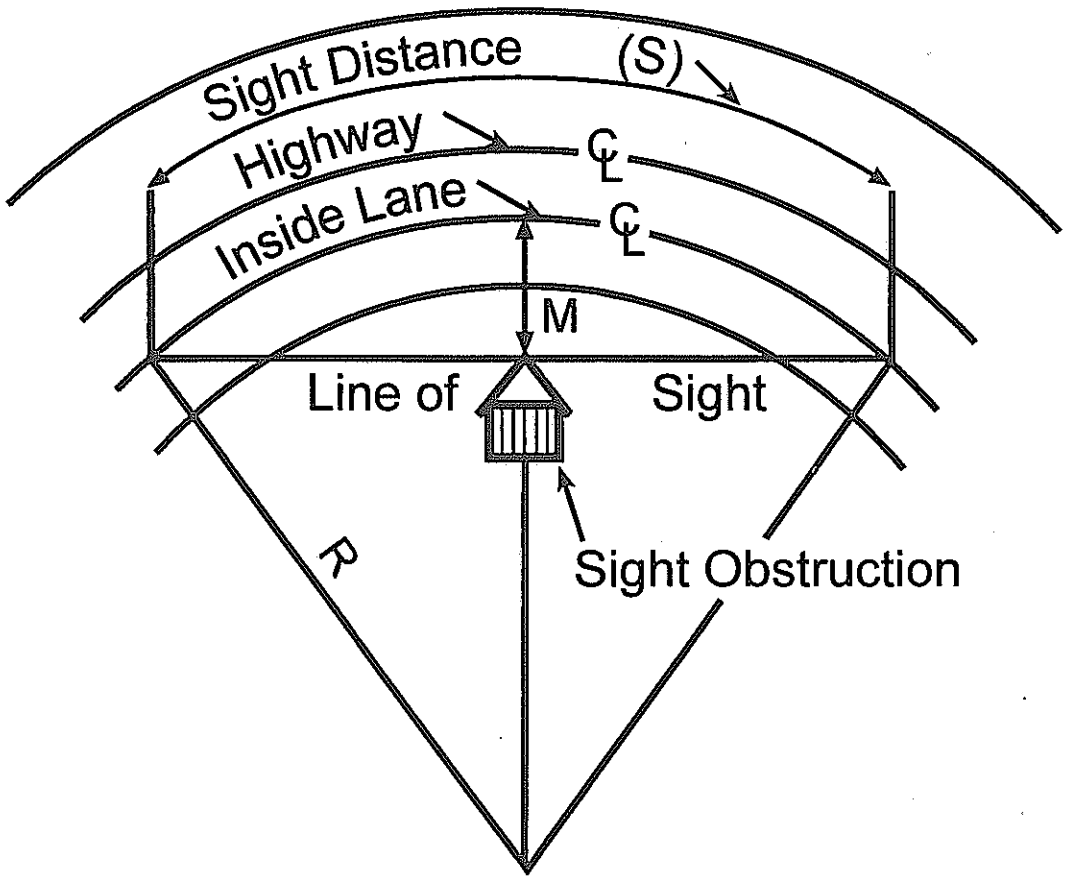


Exhibit 9. Horizontal Curve Showing Sight Distance Along the Curve and the Middle Ordinate that Defines the Maximum Unobstructed Width

Metric

All locations for 0–100 vpd and "lower risk" locations for 100–250 vpd[1]											"Higher risk" locations for 100–250 vpd and all locations for 250–450 vpd[2]										
Design speed (km/h)	Stopping sight distance (m)	Width on inside of curve clear of sight obstructions[3] (m) Radius of curvature (m)									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			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	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	30	30	–	2.2	1.1	0.6	0.2	0.1	0.1	0.0	0.0
40	35	–	3.0	1.5	0.8	0.3	0.2	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	45	–	–	2.5	1.3	0.5	0.3	0.1	0.1	0.0	50	55	–	–	3.8	1.9	0.8	0.4	0.2	0.1	0.1
60	60	–	–	–	2.2	0.9	0.5	0.2	0.1	0.1	60	70	–	–	–	3.1	1.2	0.6	0.3	0.2	0.1
70	75	–	–	–	3.5	1.4	0.7	0.4	0.2	0.1	70	90	–	–	–	5.0	2.0	1.0	0.5	0.3	0.2
80	95	–	–	–	5.6	2.3	1.1	0.6	0.3	0.2	80	110	–	–	–	7.5	3.0	1.5	0.8	0.4	0.3
90	120	–	–	–	–	3.6	1.8	0.9	0.5	0.3	90	130	–	–	–	–	4.2	2.1	1.1	0.5	0.4
100	140	–	–	–	–	4.9	2.4	1.2	0.6	0.4	100	155	–	–	–	–	6.0	3.0	1.5	0.8	0.5

US Customary

All locations for 0-100 vpd and "lower risk" locations for 100–250 vpd[1]											"Higher risk" locations for 100–250 vpd and all locations for 250–450 vpd[2]										
Design speed (mph)	Stopping sight distance (ft)	Width on inside of curve clear of sight obstructions[3] (ft) Radius of curvature (ft)									Design speed (mph)	Stopping sight distance (ft)	Width on inside of curve clear of sight obstructions[3] (m) Radius of curvature (ft)								
		50	100	200	500	1000	2000	5000	10000	20000			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	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	20	95	–	11.1	5.6	2.3	1.1	0.6	0.2	0.1	0.1
25	115	–	–	8.2	3.3	1.7	0.8	0.3	0.2	0.1	25	125	–	–	9.7	3.9	2.0	1.0	0.4	0.2	0.1
30	135	–	–	11.3	4.5	2.3	1.1	0.5	0.2	0.1	30	165	–	–	16.8	6.8	3.4	1.7	0.7	0.3	0.2
35	170	–	–	–	7.2	3.6	1.8	0.7	0.4	0.2	35	205	–	–	–	10.5	5.2	2.6	1.1	0.5	0.3
40	215	–	–	–	11.5	5.8	2.9	1.2	0.6	0.3	40	250	–	–	–	15.5	7.8	3.9	1.6	0.8	0.4
45	260	–	–	–	16.8	8.4	4.2	1.7	0.8	0.4	45	300	–	–	–	22.3	11.2	5.6	2.3	1.1	0.6
50	310	–	–	–	–	12.0	6.0	2.4	1.2	0.6	50	350	–	–	–	–	15.3	7.7	3.1	1.5	0.8
55	365	–	–	–	–	16.6	8.3	3.3	1.7	0.8	55	405	–	–	–	–	20.4	10.2	4.1	2.1	1.0
60	435	–	–	–	–	23.6	11.8	4.7	2.4	1.2	60	470	–	–	–	–	27.5	13.8	5.5	2.8	1.4

[1] "lower risk" locations are locations away from intersections, narrow bridges, railroad-highway grade crossings, sharp curves, and steep grades

[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

Exhibit 10. Design Guidelines for Sight Distance on Horizontal Curves for New Construction of Very Low-Volume Local Roads

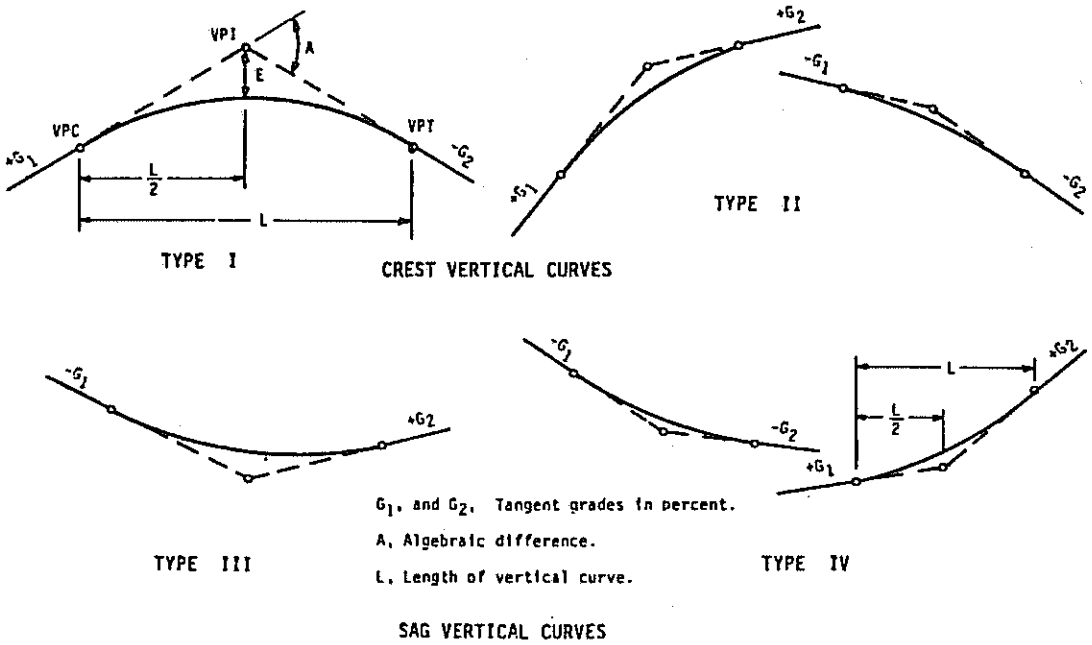


Exhibit 11. Types of Vertical Curves

Crest Vertical Curves

The major control for safe operation on crest vertical curves is the provision of ample sight distance for the roadway design speed. In new construction of very low-volume local roads, crest vertical curves should generally be designed to have at least the length that provides the stopping sight distance values presented in Exhibit 8. These lengths can be determined as shown in Equations (5) through (8):

Metric	US Customary
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, $L = \frac{AS^2}{100\left(\sqrt{2h_1}+\sqrt{2h_2}\right)^2}$ (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, $L = 2S - \frac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A}$ (6)
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.	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.

When the height of eye (h_1) and height of object (h_2) are 1,080 mm and 600 mm [3.5 ft and 2.0 ft], respectively, as used for stopping sight distance, Equations (5) and (6) become:

Metric	US Customary
When S is less than L, $L = \frac{AS^2}{658}$	When S is less than L, $L = \frac{AS^2}{2158}$ (7)
When S is greater than L, $L = 2S - \frac{658}{A}$	When S is greater than L, $L = 2S - \frac{2158}{A}$ (8)

Exhibit 12 presents the rate of vertical curvature, *K*, that will provide stopping sight distance for crest vertical curves on very low-volume local roads. The appropriate length for a vertical curve can generally be determined by multiplying the *K*-value in Exhibit 12 by the algebraic difference in grade between the adjoining tangents.

Sag Vertical Curves

There are no special guidelines for design of sag vertical curves on very low-volume local roads. Sag vertical curves should generally be designed in accordance with Chapter 5 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1).

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 very low-volume local roads to increase stopping sight distance not cost-effective except in unusual cases. Research NCHRP Report 400 (7) found that, even on higher volume roadways, accidents associated with limited sight distance are extremely rare events. Furthermore, there was no indication that lengthening of 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 very low-volume local roads than on the higher volume roads studied in NCHRP Report 400 (7).

Metric						
	All locations for 0–100 vpd and "lower risk" locations for 100-250 vpd[1]			"Higher risk" locations for 100–250 vpd and all locations for 250–400 vpd[2]		
Design speed (km/h)	Stopping sight distance (m)	Rate of vertical curvature, K[3]		Stopping sight distance (m)	Rate of vertical curvature, K[3]	
		Calculated	Design		Calculated	Design
20	15	0.3	0.5	15	0.3	0.5
30	25	0.9	1	30	1.4	2
40	35	1.9	2	40	2.4	4
50	45	3.1	4	55	4.6	5
60	60	5.5	6	70	7.4	8
70	75	8.5	9	90	12.3	13
80	95	13.7	14	110	18.4	19
90	120	21.9	22	130	25.7	26
100	140	29.8	30	155	36.5	37

US Customary						
	All locations for 0–100 vpd and "lower risk" locations for 100–250 vpd[1]			"Higher risk" locations for 100–250 vpd and all locations for 250–400 vpd[2]		
Design speed (mph)	Stopping sight distance (ft)	Rate of vertical curvature, K[3]		Stopping sight distance (ft)	Rate of vertical curvature, K[3]	
		Calculated	Design		Calculated	Design
15	65	2.0	2	65	2.0	2
20	90	3.8	4	95	4.2	5
25	115	6.1	7	125	7.2	8
30	135	8.4	9	165	12.6	13
35	170	13.4	14	205	19.5	20
40	215	21.4	22	250	29.0	29
45	260	31.3	32	300	41.7	42
50	310	44.5	45	350	56.8	57
55	365	61.7	62	405	76.0	76
60	435	87.7	88	470	102.4	103

1 "lower risk" locations are locations away from intersections, narrow bridges, railroad-highway grade crossings, sharp curves, and steep grades

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.

Exhibit 12. Guidelines for Minimum Rate of Vertical Curvature to Provide Design Stopping Sight Distance on Crest Vertical Curves for New Construction of Very Low-Volume Local Roads

Because sight distance improvements are unlikely to be cost-effective under most circumstances, the existing sight distance on a very low-volume local road may be allowed to remain in place unless there is evidence of a site-specific safety problem attributable to inadequate sight distance. If a site-specific safety problem is identified, and if the designer finds after investigation that the safety problem 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 Exhibit 8 as part of any reconstruction project undertaken. Sight distance could be increased to the full criteria presented in the AASHTO *Policy on Geometric Design of Highways and Streets* (1) 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 safety problems is found in Chapter 3 of these guidelines.

INTERSECTION SIGHT DISTANCE

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 very low-volume local roads 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 should be designed in accordance with Chapter 9 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1).

Stopping sight distance is 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, 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 safe 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 very low-volume local roads can be operated safely 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 safety.

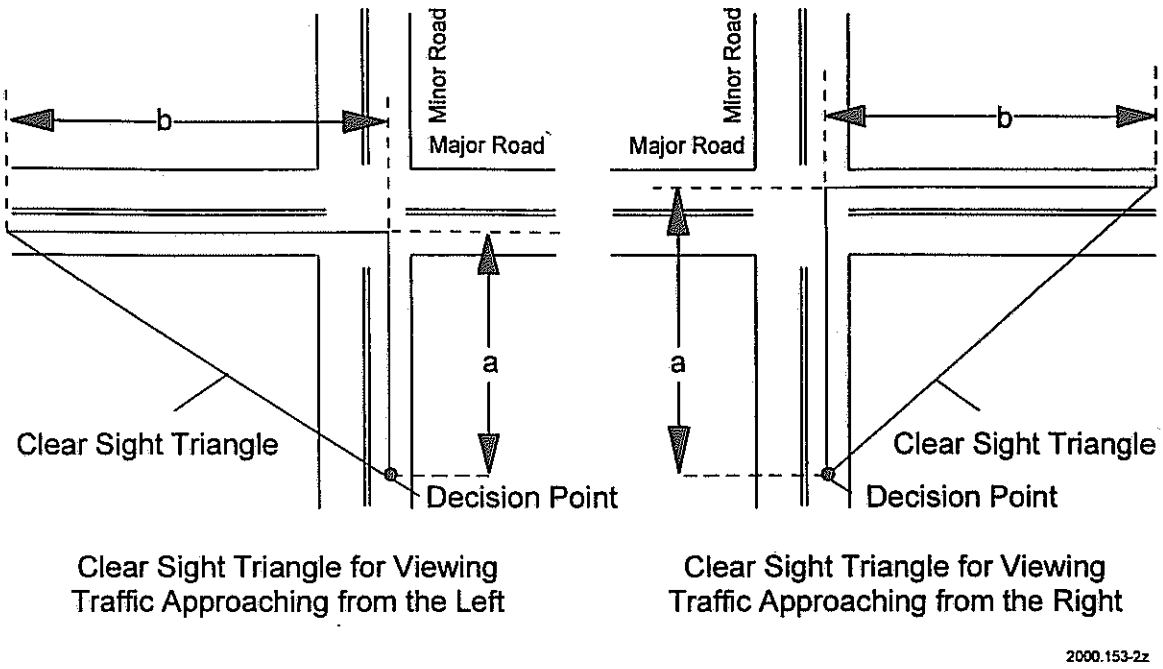
Clear Sight Triangles

Specified areas along intersection approach legs and across their included corners should be clear to 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 **(6)** that have observed driver behavior and have documented the space-time profiles and speed choices of drivers on intersection approaches.

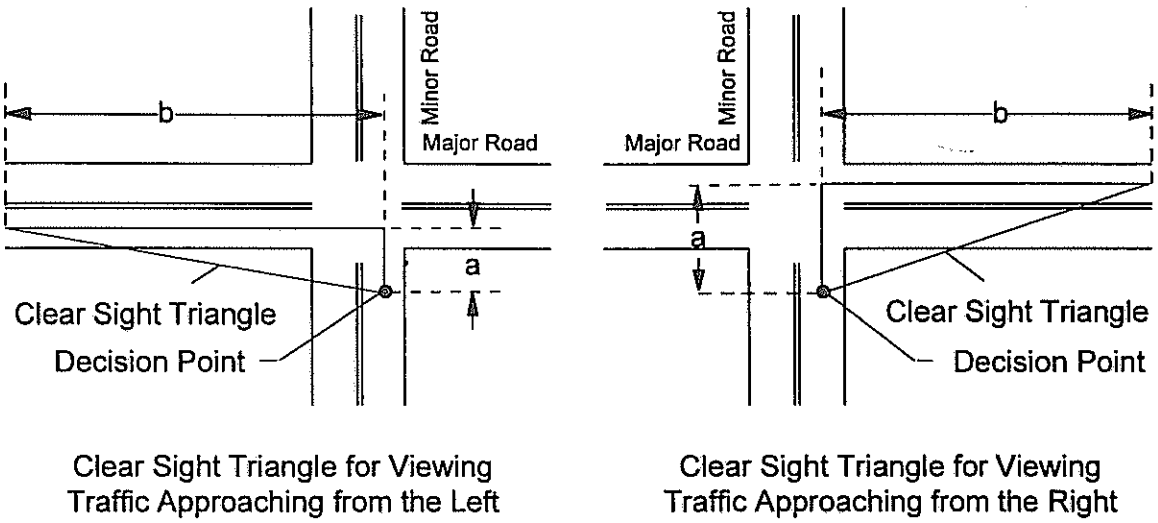
Approach Sight Triangles

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 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. Exhibit 13A 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.



A – Approach Sight Triangles



B – Departure Sight Triangles

Exhibit 13. Clear Sight Triangles for Intersection Approaches

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 Exhibit 13A 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.

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. Exhibit 13B shows typical departure sight triangles to the left and to the right. 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.

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 highway structures, roadside hardware, hedges, trees, bushes, unmowed grass, tall crops, and the should be removed or lowered, if practical. Such objects may include: buildings, parked vehicles, 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 1,080 mm [3.5 ft] above the roadway surface and that the object to be seen is also 1,080 mm [3.5 ft] above the surface of the intersecting road. This object height is based on a vehicle height of 1,330 mm [4.4 ft], which represents the 15th percentile of vehicle heights in the current passenger car population less an allowance of 250 mm [0.9 ft], 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).

New Construction

Sight distance design for newly constructed intersections at which all intersection legs are very low-volume local roads 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 *Policy on Geometric Design of Highways and Streets* **(1)** should be applied.

The sight distance design criteria for intersections between very low-volume local 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)
- Intersections with yield control on the minor road (Case C)

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 must be able to see potentially conflicting vehicles on intersecting approaches in sufficient time for the approaching driver to safely 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 require up to 2.5 s to perceive vehicles on intersecting approaches and to initiate braking.

While some perceptual tasks at intersections may require 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 require 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 **(6)** 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 1.5 m/s^2 [5 ft/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 midblock running speed during all or part of the perception-reaction time and can, therefore, where necessary, brake to a stop from a speed less than the midblock running speed.

Exhibit 14 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 Exhibit 13A. Referring to Exhibit 13A, roadway A with an 80 km/h [50 mph] design speed and roadway B with a 50 km/h [30 mph] design speed require a clear sight triangle with legs extending at least 80 m [255 ft] and 40 m [120 ft] 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 between intersections for that intersection leg.

The distances shown in Exhibit 14 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 provide an even greater margin of safety. 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 Exhibit 14 by the appropriate adjustment factor from Exhibit 15.

If the sight distances given in Exhibit 14, 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.

Metric		US Customary	
Design speed (km/h)	Sight distance (m)	Design speed (mph)	Sight distance (ft)
20	20	15	60
30	25	20	80
40	30	25	95
50	40	30	120
60	50	35	140
70	65	40	170
80	80	45	210
90	95	50	255
100	120	55	300
		60	350

Note: For approach grades greater than 3%, multiply the sight distance value by the appropriate adjustment factor from Exhibit 15.

Exhibit 14. Recommended Sight Distance Guidelines for New Construction of Intersections with No Traffic Control (Case A) (1, 6)

Metric		US Customary	
Design speed (km/h)	Sight distance (m)	Design speed (mph)	Sight distance (ft)
20	20	15	60
30	25	20	80
40	30	25	95
50	40	30	120
60	50	35	140
70	65	40	170
80	80	45	210
90	95	50	255
100	120	55	300
		60	350

Note: For approach grades greater than 3%, multiply the sight distance value by the appropriate adjustment factor from Exhibit 15.

Exhibit 15. Recommended Sight Distance Guidelines for New Construction of Intersections with No Traffic Control (Case A) (1, 6)

No departure sight triangle like that shown in Exhibit 13B is needed at an uncontrolled intersection because of the very low traffic volumes present on the intersection approaches.

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 very unlikely that another potentially conflicting vehicle will be encountered as the first vehicle departs the intersection.

Intersections with Stop Control on the Minor Road (Case B)

No approach sight triangles like those shown in Exhibit 13A 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 Exhibit 13B 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 *Policy on Geometric Design of Highways and Streets* (1), 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 Exhibit 8. For the traffic volume range from 100 to 250 vehicles per day, the sight distances in the column of Exhibit 8 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 4.4 m [14.4 ft] from the edge of the major-road traveled way **(1, 6)**.

Intersections with Yield Control on the Minor Road (Case C)

Approach sight triangles to the left and to the right like those shown in Exhibit 13A 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 *Policy on Geometric Design of Highways and Streets* (1), 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 Exhibit 8. For the traffic volume range from 100 to 250 vehicles per day, the sight distances in the column of Exhibit 8 headed "higher risk" locations should be used because this column is appropriate for application to intersections. The grade adjustment factors in Exhibit 15 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 very low-volume local roads.

Existing Roads

For improvement projects at existing intersections between very low-volume local roads, the existing intersection sight distance may generally remain in place unless there is evidence of a site-specific safety problem related to intersection sight distance. Where there is evidence of a site-specific safety problem, the intersection sight distance should be increased to at least the appropriate values shown above for new construction.

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* (2). Specific roadside design policies for higher volume local roads are also presented in Chapter 5 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1). This section presents guidelines for roadside design on very low-volume local 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 safety on very low-volume local 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 (3) indicate that run-off-road crashes on roads with very low traffic volumes 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 requires additional right-of-way acquisition which potentially has both cost and environmental concerns.

Research has found that roadside clear zones provide very little benefit, and that traffic barriers are not generally cost-effective, on roads with very low traffic volumes (**9, 10, 11**). 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 very low-volume local roads provide great flexibility to the designer in exercising engineering judgment to decide where it is appropriate to provide improved roadsides.

New Construction

Roadside design guidelines applicable to new construction of very low-volume local roads are presented below. The guidelines address both clear zone width and traffic barrier warrants and are appropriate for all functional classes of very low-volume local roads.

Clear Zone Width

The risk assessment discussed in Section 3 of this guide found that it is not generally cost-effective to provide clear zones, also known as clear recovery areas, on very low-volume local roads. Nevertheless, a clear zone of any width should provide some contribution to safety. Thus, where clear zones can be provided on very low-volume local roads at little or no additional cost, their incorporation in designs should be considered. However, major expenditures to provide clear zones will generally have only limited safety benefits and are unlikely to be cost-effective. The design guidelines for roadside clear zone width on very low-volume local roads are as follows:

1. At locations where a clear recovery area of 2 m [6 ft] or more in width can be provided at low cost and with minimum social/environmental impacts, provision of such a clear recovery area should be considered.

2. Where constraints of cost, terrain, right of way, or potential social/environmental impacts make the provision of a 2-m [6-ft] clear recovery area impractical, clear recovery areas less than 2 m [6 ft] in width may be used, including designs with 0 m [0 ft] clear recovery areas.

3. In all cases, designers should be encouraged to tailor the roadside design to site-specific conditions, considering cost-effectiveness and safety tradeoffs. For example, the use of adjustable clear zone widths, such as providing wider clear zone dimensions at sharp horizontal curves where there is a history of run-off-road crashes, or where there is evidence of vehicle encroachments such as scarring of trees or utility poles, may be appropriate. 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 2.6 m [8.5 ft] and vehicles with wide loads, such as farm equipment.

In summary, the designer should provide a clear zone as wide as practical within constraints of cost, terrain, right of way, or potential social/environmental impacts. Where provision of a clear zone is not practical, none is required. Site-specific conditions and the engineering judgment of the designer should be the two primary determinants of the appropriate clear zone width for very low-volume local roads.

Traffic Barriers

The use of guardrail or other traffic barriers to shield or protect drivers from roadside obstructions is not generally cost-effective for very low-volume local roads. This finding has been confirmed in studies by Stephens (**9**) and Wolford and Sicking (**10**). 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 very low traffic volumes. Despite the general lack of cost effectiveness for guardrail on very low-volume local roads, designers may exercise engineering judgement concerning the placement of guardrail at locations where the potential consequences of departure from the roadway are likely to be extremely severe.

Existing Roads

The roadside design guidelines for existing very low-volume local 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 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 safety problem 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 very rare on very low-volume local roads.

UNPAVED ROADS

Many low-volume local roads have unpaved surfaces. Unpaved roads are generally appropriate for all functional subclasses of very low-volume local 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 very low-volume local roads for which the cost of constructing and maintaining a paved surface would be prohibitive.

The safety of unpaved roads has been addressed in NCHRP Report 362 (**5**). This research established that crash rates are generally higher for unpaved roads than for paved roads for traffic volumes of 250 vehicles per day or more. The risk assessment by Neuman (**3**) 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 (**5**) found crash rates for unpaved roads to be lower for narrower roadway widths. Therefore, existing unpaved roads should not generally be widened as a safety measure unless there is evidence of a site-specific safety problem 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 70 km/h [45 mph] or less, but may occasionally be as high as 80 km/h [50 mph] 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 necessary 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.

Exhibit 16 presents guidelines for the minimum radius of curvature for unpaved surfaces with no superelevation for application on very low-volume local roads. The exhibit is based on the design criteria of the United States Forest Service (**11**), 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 Exhibit 17. The recommended minimum curve radii in Exhibit 16 are based on use of a side friction factor, f, in Equation (2) that is 0.2 less than the traction coefficients shown in Exhibit 17. 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, and/or with an assumption that poor surface conditions such as snow, ice, or wet pavement are not sufficiently frequent for use as

a design control. The choice of the appropriate surface condition from Exhibit 17 should be based on the engineering judgment of the designer based on site-specific conditions.

Smaller curve radii than those shown in Exhibit 16 may be used where superelevation is provided. The minimum radius of curvature for such cases can be determined with Equation (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.

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	35
30	15	20	25	40	75
40	30	35	45	65	130
50	40	50	70	100	200
60	60	75	95	145	285
70	80	100	130	195	385
US Customary					
Design speed (mph)	Minimum radius (ft) for specified traction coefficient				
	0.7	0.6	0.5	0.4	0.3
15	50	50	50	75	150
20	55	70	90	135	270
25	85	105	140	210	420
30	120	150	200	300	600
35	165	205	275	410	820
40	215	270	360	535	1070
45	270	340	450	675	1350

Source: Adapted from USFS Preconstruction Handbook (11)

Exhibit 16. Guidelines for Minimum Radius of Curvature for New Construction of Unpaved Surfaces with No Superelevation (11)

Material	Surface Condition		
	Dry	Wet	Other
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	–
Earth[a]	0.55 - 0.65	0.40 - 0.50	–
Dry, packed snow	–	–	0.20 - 0.55
Loose snow	–	–	0.10 - 0.60
Snow, lightly salted	–	–	0.29 - 0.31
Snow, lightly salted with chains	–	–	0.34
Ice, without chains	–	–	0.07 - 0.12

[a] reduce earth values by 50 percent for wet clays

Source: USFS Road Preconstruction Handbook (11)

Exhibit 17. Traction Coefficients Used in Design of Horizontal Alignment on Unpaved Roads (11)

TWO-WAY SINGLE-LANE ROADS

Two-way single-lane roads may be used in constrained locations, where traffic volumes are extremely low. Such cross sections are normally used on local roads where traffic volumes are less than 50 vehicles per day. 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 used for traffic volumes up to 100 vehicles per day. Two-way single-lane roads are designed to operate at low speeds, typically no more than 50 km/h [30 mph].

Two-way single-lane roads are often unpaved and normally have widths in the range from 3.5 to 4.0 m [11.5 to 13.0 ft]. 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 Exhibit 8. USFS guidelines recommend that turnouts be provided at regular intervals on two-way single-lane roads to allow opposing vehicles to pass one another safely (**11**). 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.

CHAPTER 5
DESIGN EXAMPLES

This chapter presents eight examples of the application of the design guidelines for very low-volume local roads presented in Chapter 4. The eight examples are hypothetical situations that illustrate how the design guidelines are intended to be applied. The general subject of the eight examples are:

- new construction of a major access road in a rural area
- resurfacing of an existing major access road in a rural area
- rehabilitation of a collector road in a rural area
- new construction of an industrial/commercial access road in a rural area
- reconstruction of a minor access road in a rural area
- new construction of an unpaved minor access road in a rural area
- new construction of an urban residential street
- reconstruction of an urban industrial/commercial access street

The specific examples are presented below.

EXAMPLE 1

A county engineer has been given the job of designing a new rural major access road with a design speed of 80 km/h [50 mph]. 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 very low-volume local 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 Classification:	rural major access road
Design Speed:	80 km/h [50 mph]
Design Traffic Volume:	350 vehicles per day

Cross Section

Guidelines for total roadway width for very low-volume local roads in rural areas are presented in Exhibit 1. Since the new roadway will be a major access road with a design speed of 80 km/h [50 mph], the county engineer selects from Exhibit 1 a total roadway width of 6.0 m [20.0 ft] for the project.

The engineer determines that the project could be built within a minimum right-of-way width of 15 m [48 ft]. 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 18 m [60 ft]. 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

Exhibit 3 presents the values of f_{max} and R_{min} used in the design of higher volume roadways. However, for the design of very low-volume local roads without substantial truck volumes, acceptable operations can be obtained with smaller curve radii than those shown in Exhibit 3. Exhibit 6 presents values of f_{max} and R_{min} that can be used in preference to those in Exhibit 3 in the design of very low-volume local roads with average daily traffic volumes of 250 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 must make sure that all curves in the horizontal alignment have a minimum radius of 230 m [760 ft] if the design is based on Exhibit 3 or a minimum radius of 150 m [615 ft] if the design is based on Exhibit 6. 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 Exhibit 3 which are based on the AASHTO *Policy on Geometric Design of Highways and Streets* (1).

Superelevation

For each individual horizontal curve, the designer selects the design superelevation based on the criteria in Chapter 3 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1), based on a design speed of 80 km/h [50 mph] 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 *Policy on Geometric Design of Highways and Streets* (1).

Stopping Sight Distance

Design Sight Distance

Exhibit 8 presents the design sight distance criteria for very low-volume local roads. Since the new roadway will have a design speed of 80 km/h [50 mph] and a projected traffic volume of 350 vehicles per day, the minimum stopping sight distance for this project is 110 m [350 ft].

Crest Vertical Curves

Exhibit 12 shows that to achieve the design sight distance of 110 m [350 ft], all crest vertical curves should be designed with a rate of vertical curvature, K, of at least 19 m [57 ft] per percent difference in grade.

Sag Vertical Curves

There are no special guidelines for design of sag vertical curves on very low-volume local roads. Therefore, the county engineer should design the sag vertical curves in accordance with Chapter 5 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1).

Horizontal Curves

The county engineer uses Equation (4) or Exhibit 10 to determine the width that should be clear of sight obstructions on the inside of each horizontal curve. The dimension, M, computed with Equation (4) or determined from Exhibit 10 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 Exhibit 13B, 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 *Policy on Geometric Design of Highways and Streets* (1). 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 110 m [350 ft], as determined from Exhibit 8. 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 2 m [6.5 ft] 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 *Policy on Geometric Design of Highways and Streets*, 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 *Policy on Geometric Design of Highways and Streets* (**1**).

EXAMPLE 2

A state highway agency is about to begin a resurfacing project on a major access road with a design speed of 90 km/h [55 mph]. 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. Very 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 very low volumes, the highway agency has concluded that it is appropriate to use the design guidelines for very low-volume local roads presented in Chapter 4.

One of the horizontal curves within the project has experienced a site-specific safety problem—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 safety 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 Classification:	rural major access road
Design Speed:	90 km/h [55 mph]
Design Traffic Volume:	200 vehicles per day

Cross Section

The existing total roadway width within the project is 6.3 m [20.7 ft]. The cross section guidelines for new construction for a rural major access road with a design speed of 90 km/h [55 mph] in Exhibit 1 indicate that a total roadway width of 6.6 m [22.0 ft] would be appropriate. However, because no site-specific safety problems attributable to cross section width have been found at the site, the existing total roadway width of 6.3 m [20.7 ft] may remain in place.

Horizontal Alignment

For improvement projects to existing roads, the guidelines suggest that for curves on very low-volume local roads with higher speeds, reconstruction without changing the existing curve geometry and cross section or making other safety improvements is acceptable if:

1. The nominal design speed of the curve is within 20 km/h or 10 mph of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific safety problem 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 safety problems at the horizontal curve with more cost-effective solutions than curve flattening and reconstruction.

Since both the accident 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 safety problem 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 safety problem 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 safety problem indicating the desirability of providing a wider roadside clear zone or guardrail. Therefore, no improvements will be made to the roadside design.

EXAMPLE 3

A state highway agency plans to rehabilitate a collector road with a design speed of 100 km/h [60 mph] in a rural area. This 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 Classification:	rural collector that qualifies for treatment as a rural major access road
Design Speed:	100 km/h [60 mph]
Design Traffic Volume:	300 vehicles per day

Cross Section

The existing total roadway width within the project over most of its length is 7.2 m [24 ft]. This exceeds the roadway width guidelines of 6.6 m [22 ft] for major access roads shown in Exhibit 1. Furthermore, there is no evidence of any site-specific safety problem related to roadway width. Therefore, the existing roadway width may remain in place.

One 3.2-km [2-mi] section of the project has a total roadway width of 6.1 m [20 ft]. While there is no evidence of an existing safety problem that would make widening desirable, the design engineer decides that this section should be widened to a total roadway width of 7.2 m [24 ft] 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 safety improvements if:

1. The nominal design speed of the curve is within 20 km/h or 10 mph of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific safety problem 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 safety problem 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 safety problem 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 safety problem indicating the desirability of providing a wider roadside clear zone or guardrail. Therefore, no improvements will be made to roadside design.

EXAMPLE 4

An engineering consultant has been hired by a township to design a new rural industrial/commercial access road with a design speed of 50 km/h [30 mph]. 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 very low-volume local 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 Classification:	rural industrial/commercial access road
Design Speed:	50 km/h [30 mph]
Design Traffic Volume:	100 vehicles per day

Cross Section

Guidelines for total roadway width for very low-volume local roads in rural areas are presented in Exhibit 1. Since the new roadway will be an industrial/commercial access road with a design speed of 50 km/h [30 mph], the consultant selects from Exhibit 1 a total roadway width of 6.8 m [22.5 ft] for the project.

Horizontal Alignment

Maximum Friction Factor and Minimum Radius

Exhibit 3 presents the values of f_{max} and R_{min} used in design of higher volume roadways. However, for the design of very low-volume roads, even those with substantial proportions of truck traffic, acceptable operations can be obtained with smaller curve radii than those shown in Exhibit 3. Exhibit 7 presents values of f_{max} and R_{min} that can be used in preference to those in Exhibit 3 in the design of very low-volume local 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 must make sure that all curves in the horizontal alignment have a minimum radius of 90 m [275 ft] if the design is based on Exhibit 3 or a minimum radius of 70 m [185 ft] if the design is based on Exhibit 7. 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 Exhibit 7 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 Exhibit 7.

Superelevation

For each individual horizontal curve, the designer selects the design superelevation based on the criteria in Chapter 3 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1) based on a design speed of 50 km/h [30 mph] 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 *Policy on Geometric Design of Highways and Streets* (1).

Stopping Sight Distance

Design Sight Distance

Exhibit 8 presents the design sight distance criteria for very low-volume local roads. Since the new roadway will have a design speed of 50 km/h [30 mph] and a projected traffic volume of 100 vehicles per day, the minimum stopping sight distance for this project is 45 m [135 ft].

Crest Vertical Curves

Exhibit 12 shows that to achieve the design sight distance of 45 m [135 ft], all crest vertical curves should be designed with a rate of vertical curvature, K, of at least 4 m [9 ft] per percent difference in grade.

Sag Vertical Curves

There are no special guidelines for design of sag vertical curves on very low-volume local roads. Therefore, the consultant designs the sag vertical curves in accordance with Chapter 5 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1).

Horizontal Curves

The consultant uses Equation (4) to determine the width that should be clear of sight obstructions on the inside of each horizontal curve. The dimension, M, computed with Equation (4) or determined from Exhibit 10 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 Exhibit 13A, 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 Exhibit 14. The leg extending along the road being constructed, with a 50 km/h [30 mph] design speed, should be at least 40 m [120 ft]. The leg extending along the intersecting roadway will be determined from Exhibit 14 based on the design speed of that roadway.

Departure Sight Triangle

Departure sight triangles, like those shown in Exhibit 13B, 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 45 m [135 ft], as determined from Exhibit 8. The length of the departure sight triangle along the crossroad approach should be 4.4 m [14.4 ft].

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 *Policy on Geometric Design of Highways and Streets* (1).

EXAMPLE 5

A rural county is about to begin a reconstruction project on a minor access road with a design speed of 40 km/h [25 mph]. 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 Classification:	rural minor access road
Design Speed:	40 km/h [25 mph]
Design Traffic Volume:	100 to 125 vehicles per day or less

Cross Section

There is no evidence of any site-specific safety problems. 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 higher speed very low-volume local 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 km/h or 10 mph of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific safety problem 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 safety problem 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 safety problem 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 safety problem 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, these trees constitute a safety hazard. Therefore, the county engineer decides that these trees should remain in place. However, the county engineer does find that 5 m [16 ft] 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.

EXAMPLE 6

A rural township is planning to construct an unpaved rural minor access road on a new alignment. The design speed will be 60 km/h [40 mph] 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 Classification:	rural minor access road
Design Speed:	60 km/h [40 mph]
Design Traffic Volume:	90 vehicles per day

Cross Section

The total roadway width selected for the road is 5.4 m [18 ft] based on the guidelines presented in Exhibit 1.

Horizontal Alignment

The surfacing material selected for the roadway is loose gravel with an expected traction coefficient of 0.5 under wet conditions, based on Exhibit 17. A traction coefficient of 0.5 corresponds to a side friction factor, *f*, of 0.3. Exhibit 16 indicates that the appropriate minimum radius of curvature for a design speed of 60 km/h [40 mph] and a traction coefficient of 0.5 is 95 m [360 ft]. This minimum radius applies to curves with no superelevation. If superelevation of 4 percent is provided, the equivalent minimum radius, determined from Equation (2), curves could be built with a minimum radius of 85 m [315 ft]. In fact, the sharpest curve designed by the consultant for the project has a radius of 150 m [492 ft].

Stopping Sight Distance

Design Sight Distance

Exhibit 8 presents the design sight distance guidelines for very low-volume local roads. Since the roadway will have a design speed of 60 km/h [40 mph] and a design traffic volume under 100 vehicles per day, the minimum stopping sight distance for this roadway should be 60 m [215 ft].

Crest Vertical Curves

Exhibit 12 shows that to achieve the design sight distance of 60 m [215 ft], all crest vertical curves should be designed with a rate of vertical curvature, K, of at least 6 m [22 ft] per percent difference in grade.

Sag Vertical Curves

There are no special guidelines for sag vertical curves on very low-volume local roads. Therefore, the engineer designs the sag vertical curves in accordance with Chapter 5 of the AASHTO *Policy on Geometric Design of Highways and Streets* (1).

Horizontal Curves

The engineer should use Equation (4) on Exhibit 10 to determine the width that should be clear of sight obstructions on the inside of each horizontal curve. The dimension, M, computed

with Equation (4) as determined from Exhibit 10 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 5.4-m [18-ft] roadway should be measured from a point in the roadway 1.35 m [4.5 ft] from its inside edge.

Intersection Sight Distance

The 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 Exhibit 14. 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 *Policy on Geometric Design of Highways and Streets* **(1)**.

Roadside Design

Clear Zone Width

While no specific minimum clear zone is needed, the engineer has found that a 2 m [6.5 ft] 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 *Policy on Geometric Design of Highways and Streets* **(1)**.

EXAMPLE 7

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 60 km/h [40 mph] 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 very low-volume local 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 Classification:	urban residential street
Design Speed:	60 km/h [40 mph]
Design Traffic Volume:	150 vehicles per day

Cross Section

Guidelines for total roadway width for urban residential streets are presented in Exhibit 2. These widths incorporate consideration of access for fire trucks and other emergency vehicles and apply to streets where parking is permitted on both sides of the roadway. While there will not be any marked parking spaces on the proposed residential street, parking will be permitted. The suburban charter of the development represents a low development density. Therefore, the traffic engineer determines from Exhibit 2 that a total roadway width of 8.2 m [27.0 ft] is appropriate for the project. The developer has recommended a roadway width of 7.6 m [25.0 ft], so the engineer requests that this be increased to 8.2 m [27.0 ft].

Horizontal Alignment

Maximum Friction Factor and Minimum Radius

Exhibit 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 Exhibit 3, whenever practical. However, on streets with design speeds of 60 km/h [40 mph] or less, the design criteria in Exhibit 4 may be used in preference to Exhibit 3.

If constrained conditions were present, the design could use a minimum radius of curvature of 125 m [490 ft] with the city's maximum superelevation rate, e_{max}, of 4 percent, based on Exhibit 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 Exhibit 3 which are based on the AASHTO *Policy on Geometric Design of Highways and Streets* (1). Therefore, the city engineer selects from Exhibit 3 a minimum radius of 150 m [565 ft], corresponding to a design speed of 60 km/h [40 mph] 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 221 m [724 ft]. 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 *Policy on Geometric Design of Highways and Streets* (1) for a design speed of 60 km/h [40 mph] 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 *Policy on Geometric Design of Highways and Streets* (1).

Stopping Sight Distance

Design Sight Distance

Exhibit 8 presents the design sight distance guidelines for very low-volume local roads. Since the new roadway will have a design speed of 60 km/h [40 mph] and a projected traffic volume of 150 vehicles per day, the minimum stopping sight distance for this project should be 70 m [250 ft] near intersections and 60 m [215 ft] away from intersections.

Crest Vertical Curves

Exhibit 12 shows that to achieve the design sight distance of 60 m [215 ft], all crest vertical curves should be designed with a rate of vertical curvature, K, of at least 6 m [22 ft] 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 very low-volume local 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 *Policy on Geometric Design of Highways and Streets* (1).

Horizontal Curves

The city traffic engineer uses Equation (4) to determine the width that should be clear of sight obstructions on the inside of each horizontal curve. The dimension, M, computed with Equation (4) or determined from Exhibit 10 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 Exhibit 13B, 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 *Policy on Geometric Design of Highways and Streets* (1).

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 *Policy on Geometric Design of Highways and Streets* (1).

EXAMPLE 8

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/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 60 km/h [35 mph]. 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 Classification:	urban industrial/commercial access street
Design Speed:	60 km/h [35 mph]
Design Traffic Volume:	225 to 250 vehicles per day

Cross Section

The existing unpaved street width is 6.8 m [22.5 ft], which is equal to the recommended cross section width for an industrial/commercial access street with a 60 km/h [35 mph] design speed as shown in Exhibit 1.

There is no evidence of any site-specific safety problems 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 higher speed very low-volume local 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 km/h or 10 mph of the design or operating speed of the roadway.
2. There is no clear evidence of a site-specific safety problem associated with the curve.

The city engineer determines that both requirements 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 safety problem 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 safety problem 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 safety problem indicating the desirability of providing a roadside clear zone or a guardrail. Therefore, no improvements will be made to the roadside design.

REFERENCES

1. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets*. Washington, D.C.: 2001.
2. American Association of State Highway and Transportation Officials. *Roadside Design Guide*. Washington, D.C.: January 1996.
3. Neuman, T. R. "Design Guidelines for Very Low-Volume Local Roads (< 400 ADT)," final report of NCHRP Project 20-7(75), CH2M Hill Chicago, Illinois: May 1999.
4. Transportation Research Board. *Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation*, Special Report 214. Washington, D.C.: 1987.
5. Zegeer, C. V., R. Stewart, F. Council, and T. R. Neuman, *Roadway Widths for Low-Traffic-Volume Roads*, NCHRP Report 362, Transportation Research Board. Washington, D.C.: 1994.
6. Harwood, D. W., J. M. Mason, R. E. Brydia, M. T. Pietrucha, and G. L. Gittings, *Intersection Sight Distance*, NCHRP Report 383, Transportation Research Board. Washington, D.C.: 1996.
7. Fambro, D., K. Fitzpatrick, and R. J. Koppa, *Determination of Stopping Sight Distances*, NCHRP Report 400, Transportation Research Board. Washington, D.C.: 1997.
8. Zegeer, C. V., R. Stewart, D. Reinfurt, F. M. Council, T. Neuman, E. Hamilton, T. Miller, and W. Hunter, *Cost Effective Geometric Improvements for Safety Upgrading of Horizontal Curves*, Report No. FHWA-RD-90-021. Federal Highway Administration. McLean, VA: October 1991.
9. Stephens, L. B., "Guardrail Warrants for Low-Volume Roads," in Transportation Research Circular 416, *Issues Surrounding Highway and Roadside Safety Management*. Washington, D.C.: October 1993.
10. Wolford, D., and D. L. Sicking, "Guardrail Need: Embankments and Culverts," *Transportation Research Record 1599*. Transportation Research Board. Washington, D.C.: 1997.
11. United States Forest Service. *Road Preconstruction Handbook*, Publication FSH 7709.56. U.S. Department of Agriculture. May 1987.
12. Transportation Association of Canada, *Manual of Geometric Design Standards for Canadian Roads*. Ottawa, 1986.
13. Institute of Transportation Engineers, *Recommended Guidelines for Subdivision Streets*. Washington, D.C.: 1993.