63

Geometric Design

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63.1 Introduction

Geometric design of highways refers to the design of the visible dimensions of such features as horizontal and vertical alignments, cross sections, intersections, and bicycle and pedestrian facilities. The main objective of geometric design is to produce a highway with safe, efficient, and economic traffic operations while maintaining esthetic and environmental quality. Geometric design is influenced by the vehicle, driver, and traffic characteristics. The temporal changes of these characteristics make geometric design a dynamic field where design guidelines are periodically updated to provide more satisfactory design.

Policies on highway geometric design in the United States are developed by the American Association of State Highway and Transportation Officials (AASHTO). These policies represent design guidelines agreed to by the state highway and transportation departments and the Federal Highway Administration (FHWA). Guidelines for highway geometric design are presented in *A Policy on Geometric Design of Highways and Streets* [AASHTO, 2001], which is based on many years of experience and research. Repeated citation to AASHTO throughout this chapter refers to this policy. In Canada, geometric design guidelines are presented in the *Geometric Design Guide for Canadian Roads* [TAC, 1999], which is published by the Transportation Association of Canada (TAC).

This chapter discusses the fundamentals of highway geometric design and their applications and is divided into four main sections: fundamentals of geometric design, basic design applications, special design applications, and emerging design concepts. It draws information mostly from the AASHTO policy and TAC guide and provides supplementary information on more recent developments. Since geometric design is a major component in both the preliminary location study and the final design of a proposed highway, it is useful to describe first the highway design process.

Design Process

The design process of a proposed highway involves preliminary location study, environmental impact evaluation, and final design. This process normally relies on a team of professionals, including engineers, planners, economists, sociologists, ecologists, and lawyers. Such a team may have responsibility for addressing social, environmental, land-use, and community issues associated with highway development.

Preliminary Location Study

The preliminary location study involves collecting and analyzing data, locating feasible routes, determining preliminary horizontal and vertical alignments for each, and evaluating alternative routes to select the best route. The types of data required are related to the engineering, social and demographic, environmental, and economic characteristics of the area. Examples of such data are topography, land-use pattern, wildlife types, and unit costs of construction. A preliminary study report is prepared and typically includes a general description of the proposed highway, a description of alternative locations and designs, projected traffic volumes and estimated total costs, an economic and environmental evaluation, and a recommended highway location. Before the project is approved, it is common to hold public hearings to discuss the preliminary study and environmental impacts.

Environmental Evaluation

Highway construction may impact the environment in a number of areas, including air quality, water quality, noise, wildlife, and socioeconomics. For example, highways may cause loss or degradation of a unique wildlife habitat and changes to migratory patterns. Socioeconomic impacts include displacement of people and businesses, removal of historically significant sites, and severance of the interpersonal ties of displaced residents to their former community. It is therefore essential that environmental impacts of alternative highway locations be fully evaluated.

Provisions of the National Environment Policy Act of 1969 require that an environmental impact statement (EIS) be submitted for any project affecting the quality of the environment. The EIS must describe the environmental impacts of the proposed action, both positive and negative; probable unavoidable adverse environmental impacts; secondary environmental impacts such as changes in the pattern of social and economic activities; analysis of short- and long-term impacts; irreversible and irretrievable commitments of resources; and public and minority involvement. Chapter 8, Section 10, provides more details on the environmental process.

Final Design

The final design involves establishing the design details of the selected route, including final horizontal and vertical alignments, drainage facilities, and all items of construction. The design process has been revolutionized by advanced photogrammetric and computer techniques. For example, designers now can have a driver's eye view of a proposed highway alignment displayed on a monitor and readily examine the effects of alignment refinements. Further details on the design process are found in Garber and Hoel [2001].

63.2 Fundamentals of Geometric Design

Geometric design involves a number of fundamentals and concepts that guide and control the manner in which a highway is designed. These include highway types, design controls, sight distance, and simple highway curves.

Highway Types

Classification of highways into functional classes is necessary for communication among engineers, administrators, and the general public. The functional classification system facilitates grouping roads that require the same quality of design, maintenance, and operation. The system also facilitates the logical assignment of responsibility among different jurisdictions, and its structure of the design guidelines is readily understood.

The highway functional classes, adopted separately for urban and rural areas, are locals, collectors, and arterials. The principal arterial system includes freeways and other principal arterials. The two major considerations in the functional classification system are travel mobility and land access. Locals emphasize the land access function; collectors provide a balanced service for both functions; and arterials emphasize the mobility function. Design guidelines for locals, collectors, arterials, and freeways, in both urban and rural locations, are presented by AASHTO in Chapters 5 through 8, respectively. Details on functional system characteristics are found in FHWA [1989].

Design Controls

The major controls that influence the geometric design of highways include topography, the design vehicle, driver performance, traffic characteristics, highway capacity, access control and management, the pedestrian, bicycle facilities, and safety. Other controls such as esthetics, environment, economics, and public concerns are important but are reflected in either the preceding major controls or the preliminary location study.

Design Vehicle

A design vehicle is a vehicle with representative weight, physical dimensions, and operating characteristics, used to establish highway design controls for accommodating vehicles of designated classes. Each design vehicle has larger dimensions and a larger minimum turning radius than most vehicles in its class. Four general classes of design vehicles have been established: passenger cars, buses, trucks, and recreational vehicles. The dimensions of 20 design vehicles within these general classes are given by AASHTO. The design vehicle selected for geometric design is the largest vehicle likely to use the highway with considerable frequency or a vehicle with special characteristics appropriate to a particular intersection for determining the radii at intersections and the radii of **turning roadways**. A typical minimum turning path for a single-unit (SU) truck design vehicle is shown in Fig. 63.1. Other vehicle characteristics such as acceleration and braking capabilities, the driver's eye height, and vehicle headlights also affect many geometric design features.

Driver Performance

Highways should be designed to be compatible with driver capabilities and limitations. Information about the performance of the drivers (how they interact with the highway and its information) is useful in highway design and operations. Since it is not generally possible to reduce errors caused by innate driver deficiencies, a "forgiving" design that lessens the consequences of failure should be implemented. In addition, a positive guidance approach should be applied to design. Here are some examples:

- The design should focus a driver's attention on the safety-critical elements by providing clear sight lines and good visual quality.
- The design should take into account the longer reaction time required for complex decisions by providing adequate decision sight distance.
- On high-speed facilities, guidance activities should be simplified because speed reduces the visual field, restricts peripheral vision, and limits the time available to process information.

Another important means to aid driver performance is the development of designs in accordance with driver expectancies. Detailed information on driver attributes, driving tasks, and information handling can be found in the FHWA report *A User's Guide to Positive Guidance* [Alexander and Lunenfeld, 1990].

Traffic Characteristics

Traffic characteristics include traffic volume, directional distribution, traffic composition, and speed. Design volume and composition determine the highway type, required roadway width, and other geometric

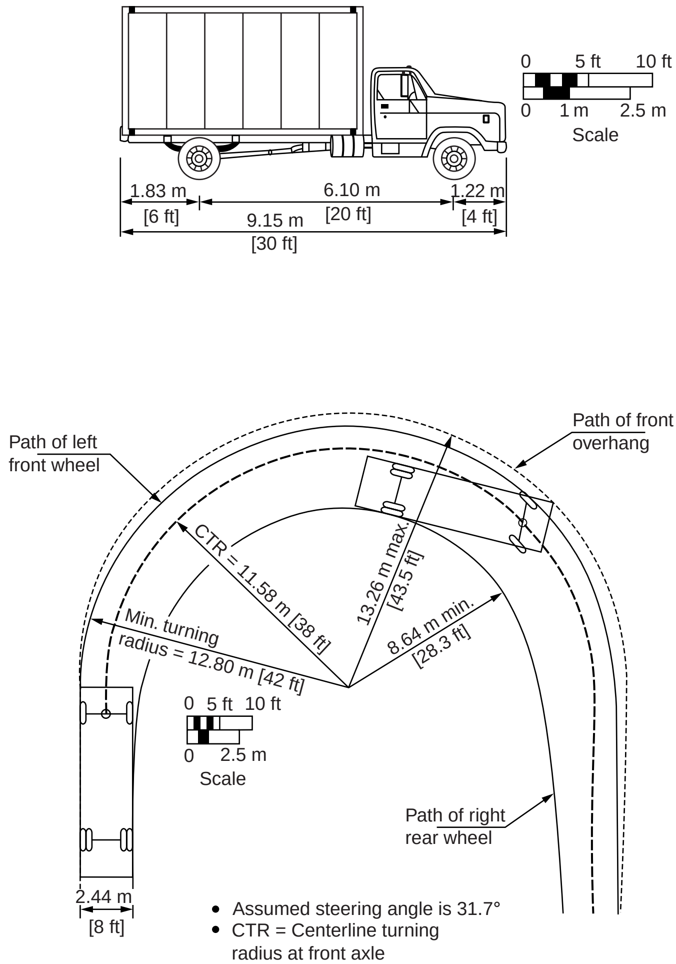


FIGURE 63.1 Minimum turning path for single-unit (SU) truck design vehicle. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

features. The basic measure of the traffic demand for a highway is the **average daily traffic** (ADT). This measure is used for selecting geometric design guidelines for local and collector roads. For other highways, the design hourly volume (DHV), a two-way volume, is used and is generally defined as the **30th highest hour volume** of a designated year. The ratio of the DHV and ADT, *P*, varies only slightly from year to year. For design of a new highway, *P* can be determined using existing traffic volumes of similar highways. The typical range of *P* is 12 to 18% for rural highways and 8 to 12% for urban highways.

For two-lane highways, the DHV is the total traffic in both directions. For multilane highways, the directional distribution of traffic during the design hour should be determined. The directional DHV can then be calculated by multiplying ADT by *P* and then by the percentage of traffic in the peak direction during the design hour. For recreational routes, in practice the DHV is selected as 50% of the volume expected to occur during the few highest hours of the design year. Other volume characteristics required for the design year include **peak-hour factor** and the percentages of trucks, buses, and recreational vehicles in the design-hour volume.

Design speed is a selected speed used to determine various geometric design features of the roadway. Selection of design speed is influenced by topography, adjacent land use, highway functional classification, and anticipated **operating speed.** Nearly all geometric design elements are directly or indirectly influenced by design speed.

TABLE 63.1 Guidelines for Selection of Design Levels of Service

Functional Class	Appropriate Level of Service for Specified Combinations of Area and Terrain Type			
	Rural Level	Rural Rolling	Rural Mountainous	Urban and Suburban
Freeway	B	B	C	C
Arterial	B	B	C	C
Collector	C	C	D	D
Local	D	D	D	D

Source: American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.

Highway Capacity

The required number of lanes of a highway depends on the DHV and the **level of service** intended for the design year. The *Highway Capacity Manual* [TRB, 2000] defines six levels of service ranging from level-of-service A (least congested) to level-of-service F (most congested). Table 63.1 shows the AASHTO-recommended design levels of service for different highway classes and locations. These levels of service are based on criteria for acceptable degrees of congestion.

Access Control and Management

Access control refers to the interference regulations of public access rights to and from properties on the roadside. These regulations include full control of access, partial control of access, and access management. Full-controlled access facilities (such as freeways) have no at-grade crossings and have access connections only with selected roads. With partial control of access, preference is given to through traffic to an extent, but there may be some at-grade crossings and driveway connections. Partial access control can be achieved by driveway permits, zoning restrictions, and frontage roads. The extent of access control is a significant factor in defining the functional type of a highway.

Access management, a new element of road design that applies to all types of roads, involves providing (managing) access to land development while simultaneously preserving the flow of traffic (safety, capacity, and speed) on the surrounding road system. It views the roadway and its surrounding activities as part of a system with the goal of coordinating the planning and design of each activity. For more details on access management, see Koepke and Levinson [1992].

Pedestrian

Interaction of pedestrians with traffic is a major consideration in highway planning and design. Pedestrian facilities include sidewalks, crosswalks, curb ramps for the handicapped, and grade separations. Sidewalks are usually provided in urban areas and in rural areas with high pedestrian concentrations, such as schools, local businesses, and industrial plants. Pedestrian crosswalks are provided at intersections and at midblocks. For guidance on pedestrian crosswalk marking, refer to the *Manual on Uniform Traffic Control Devices* (MUTCD). Curb ramps for the handicapped should be provided at all intersections that have curbs and sidewalks and at midblock pedestrian crossings. Because these crossings are generally unexpected by drivers, warning signs and adequate visibility should be provided. Pedestrian grade separations are necessary when pedestrian and traffic volumes are high or where there is abnormal inconvenience to pedestrians, such as at freeways. Design issues on safe accommodation of pedestrians are addressed in the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* [AASHTO, 2002].

Bicycle Facilities

Design of bicycle facilities is an important consideration in highway design. Design measures to enhance safety for bicycle traffic on existing highways include paved shoulders, wider outside traffic lanes, adjustment of manhole covers to pavement surface, and provision of a smooth riding surface. The highway system can also be supplemented by providing specifically designated bikeways. Important elements of bikeway design include design speed, bikeway width, superelevation, turning radii, grade, stopping sight distance, and vertical curves. The *Guide for the Development of New Bicycle Facilities* [AASHTO, 1999], in conjunction with the MUTCD [FHWA, 1988], provides guidance for bikeway planning and design.

Safety

Safety is a major consideration in the design of nearly all elements of highway geometric design, including horizontal and vertical alignments, cross sections, roadsides, traffic control devices, and intersections. Safety must be reflected not only in new highway and major reconstruction projects but also in the resurfacing, restoration, and rehabilitation (RRR) projects. AASHTO stresses the importance of establishing a safety evaluation program to identify safety hazards, evaluate the effectiveness of alternative improvements, and allocate available funds to the most effective uses. The TAC guide includes explicit evaluation of safety (collision frequency–design parameter relationships) for some highway elements, such as horizontal alignment, vertical alignment, and truck climbing lanes [TAC, 1999].

Sight Distance

Sight distance is the length of the roadway ahead that is visible to the driver. It is a fundamental design element in the safe and efficient operation of a highway. Five basic types of sight distances must be considered in design: (1) stopping sight distance (SSD), applicable on all highways; (2) passing sight distance (PSD), applicable only on two-lane highways; (3) decision sight distance (DSD), needed at complex locations; (4) preview sight distance (PVSD), applicable to horizontal curves, especially those combined with vertical curves; and (5) intersection sight distance (ISD), needed for all types of intersections. In addition, a special type of sight distance (called head-on sight distance) that may be needed when parking occurs on both sides of a residential street has been addressed by Gattis [1991].

Stopping Sight Distance

Stopping sight distance is the distance that enables a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. The SSD in feet is computed by

$$SSD = 1.47Vt + \frac{V^2}{30\left[\left(a/32.2\right) + G\right]} \tag{63.1}$$

where t = the brake reaction time (sec)
V = the design speed (mph)
a = the deceleration rate (ft/sec^2)
G = the percent of grade divided by 100 (positive for upgrade and negative for downgrade)

The recommended design criterion for brake reaction time is 2.5 sec, which exceeds the 90th percentile of reaction time for all drivers. The recommended design deceleration rate is 11.2 ft/sec^2, which is the comfortable deceleration rate for most drivers on wet surfaces. Design values of SSD for level grades ($G = 0$) are shown in Table 63.2. These values are used for such application as vertical curve design, intersection geometry, and placement of traffic control devices. In Canada, SSD is based on the coefficient of friction between the tires and the roadway (rather than deceleration rate) and assumed operation speed range for each design speed [TAC, 1999].

The SSD values of Table 63.2 are based on passenger car operation and do not explicitly consider truck operation. Trucks need longer stopping distances than passenger cars. However, truck drivers can see substantially farther beyond vertical obstruction than passenger cars, and this factor tends to balance the additional braking lengths required for trucks. Therefore, separate SSD values for trucks and passenger cars are not generally used in highway design. One exception is the case of horizontal sight restrictions, where the greater eye height of truck driver is of little value. For this case, it is desirable to provide an SSD that exceeds the values of Table 63.2.

TABLE 63.2 Design Requirements for Stopping and Passing Sight Distances

Design Speed (mph)	Stopping Sight Distance (ft) Calculated[a]	Stopping Sight Distance (ft) Design	Design Speed (mph)	Passing Sight Distance (ft) Passed Vehicle Speed (mph)	Passing Vehicle Speed (mph)	Calculated[b]	Design
20	111.9	115	20	18	28	707	710
30	196.7	200	30	26	36	1088	1090
40	300.6	305	40	34	44	1470	1470
50	423.8	425	50	41	51	1832	1835
60	566.0	570	60	47	57	2133	2135
70	727.6	730	70	54	64	2479	2480
80	908.3	910	80	58	68	2677	2680

[a] Values are calculated using Eq. (63.1).
[b] Values are based on Fig. 63.3.
Source: American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.

Example 63.1

Compute the SSD for a highway with a 60-mph design speed and G = 0. From Eq. (63.1),

$$SSD = 1.47 ¥ 2.5 ¥ 60 + \frac{(60)^2}{30(11.2/32.2)} = 566.0 \text{ ft}$$

which is the same as the computed value in Table 63.2.

Passing Sight Distance

Passing sight distance is the distance required for a vehicle to overtake a slower moving vehicle safely on a two-lane highway. The AASHTO model is based on certain assumptions for traffic behavior and considers PSD as the sum of four distances (Fig. 63.2): (1) distance during perception and reaction time, and during the initial acceleration of the passing vehicle to the encroachment point on the left lane (d_1); (2) distance traveled while the passing vehicle occupies the left lane (d_2); (3) distance between the passing vehicle at the end of its maneuver and the opposing vehicle (d_3); and (4) distance traversed by an opposing

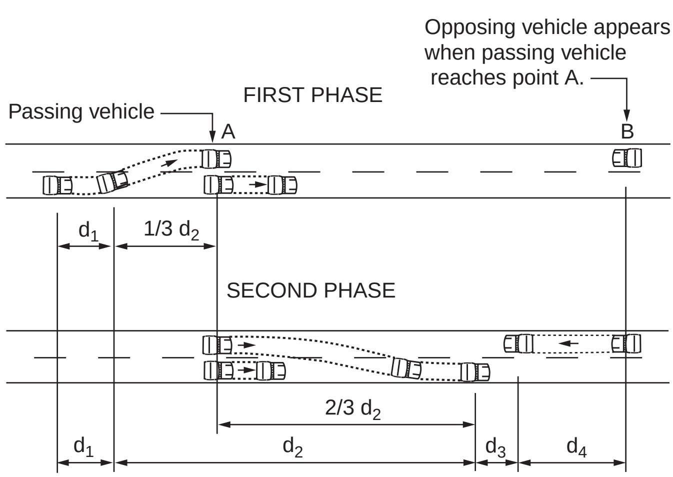


FIGURE 63.2 Elements of passing sight distance for two-lane highways. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

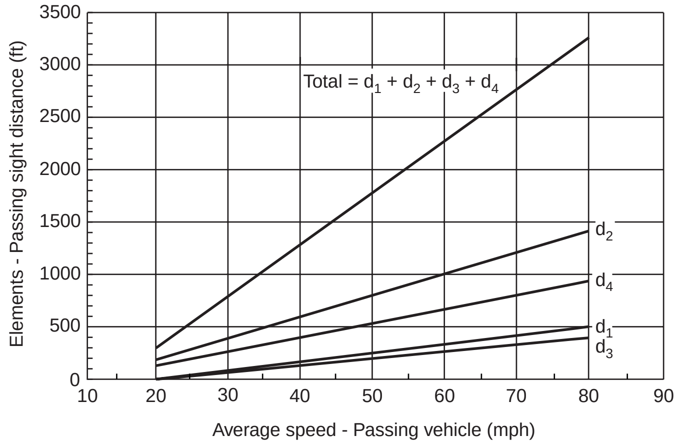


FIGURE 63.3 Total passing sight distance and its components — two-lane highways. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

vehicle for two-thirds of the time the passing vehicle occupies the left lane (d_4). The calculations of d_1 through d_4 are described in AASHTO, and the sum of the four elements is shown in Fig. 63.3.

The design values of PSD are shown in Table 63.2. The speed of the passed vehicle is assumed to be the **average running speed**, while the speed of the passing vehicle is 10 mph greater. The design values apply to a single passing only. For passing maneuvers on upgrades, the passing sight distance should be greater than the design values shown in Table 63.2. However, specific adjustments for design use are currently unavailable.

Decision Sight Distance

Decision sight distance is required at complex locations to enable drivers to maneuver their vehicles safely rather than stop. It is the distance required for a driver to detect an unexpected hazard, recognize the hazard, decide on proper maneuvers, and execute the required action safely. Examples of complex locations where provision of DSD is desirable include complex interchanges and intersections, toll plazas, lane drops, and areas where sources of information (such as signs, signals, and traffic control devices) compete. Design values for DSD, based on empirical data, are shown in Table 63.3. Since decision sight distance affords drivers sufficient length to maneuver their vehicles, its value is much greater than the stopping sight distance.

The decision sight distance is computed by

$$\text{DSD} = 1.47\ Vt + 1.075\left(V^2/a\right) \qquad \left(\text{for maneuvers A and B}\right) \tag{63.2}$$

$$\text{DSD} = 1.47\ Vt \qquad \left(\text{for maneuvers C, D, and E}\right) \tag{63.3}$$

where t = the premaneuver time for maneuvers A and B or the total premaneuver and maneuver time for maneuvers C to E (sec), based on the notes in Table 63.3
V = the design speed (mph)
a = the deceleration rate (ft/sec^2)

Note that for maneuvers A and B in Table 63.3, the premaneuver time is increased above that of SSD (2.5 sec) to allow the driver additional time to detect and recognize the roadway and traffic environment, identify alternative maneuvers, and initiate a response at critical locations. The braking distance from the design speed is then added to the premaneuver component, as noted in Eq. (63.2). For maneuvers C to E, the braking component is replaced with a maneuver distance based on maneuver times between

TABLE 63.3 Design Requirements for Decision Sight Distance

Design Speed (mph)	Decision Sight Distance (ft)				
	Avoidance Maneuver				
	A	B	C	D	E
30	220	490	450	535	620
40	330	690	600	715	825
50	465	910	750	890	1030
60	610	1150	990	1125	1280
70	780	1410	1105	1275	1445
80	970	1685	1260	1455	1650

Avoidance maneuvers:
- A: Stop on rural road (t = 3.0 sec)
- B: Stop on urban road (t = 9.1 sec)
- C: Speed, path, or direction change on rural road (t varies from 10.2 to 11.2 sec)
- D: Speed, path, or direction change on suburban road (t varies from 12.1 to 12.9 sec)
- E: Speed, path, or direction change on urban road (t varies from 14.0 to 14.5 sec)

Source: American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.

3.5 and 4.5 sec that decrease with increasing speed. Where it is not feasible to provide DSD, designers should move the location or use suitable traffic control devices to provide advance warning of the conditions to be encountered.

Preview Sight Distance

Preview sight distance is the distance required by a driver to perceive a horizontal curve and properly react to it. AASHTO implicitly recognizes PVSD by recommending that sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve or near the low point of a pronounced sag vertical curve. The preview sight distance has been suggested by Gattis and Duncan [1995] for horizontal curves. For three-dimensional alignments, PVSD is the sum of two components (Fig. 63.4): the tangent component, S_1, and the curve component, S_2 [Hassan and Easa, 2000]. The tangent component is the distance required for the driver to react and adjust the speed before reaching the curve. The curve component is the distance on the horizontal curve required for the driver to detect its existence. Preliminary design values of PVSD have been established.

Intersection Sight Distance

Intersection sight distance is the distance provided at an intersection to allow approaching vehicles (at an uncontrolled or yield-controlled intersection) to see any potentially conflicting vehicles in sufficient time to slow or stop safely and to allow stopped vehicles (at a stop or signal-controlled intersection) to enter or cross the intersection safely. Details on the calculation of intersection sight distance are presented later in this chapter.

Design Heights for Sight Distances

The AASHTO design driver's eye height and object height used for measuring various sight distances are shown in Table 63.4. Sight distances are measured from a 3.5-ft driver's eye height to a 2.0-ft object height for SSD and DSD and a 3.5-ft object height for PSD and ISD. The object height of 3.5 ft is based on a design vehicle height of 4.35 ft, less an allowance of 10 in. to allow another driver to recognize the vehicle

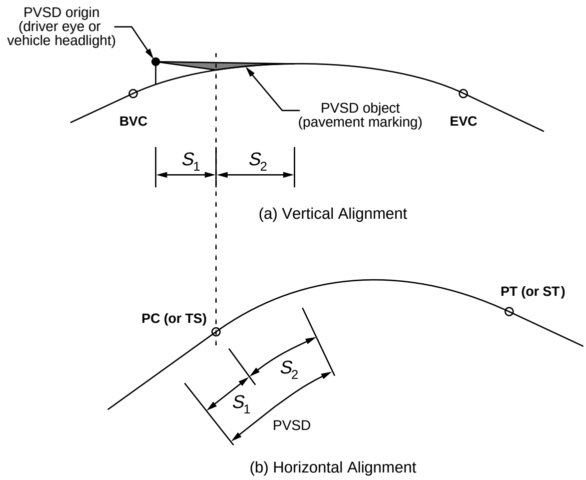


FIGURE 63.4 Illustration of preview sight distance. (From Hassan, Y. and Easa, S.M., *J. Transp. Eng. ASCE*, 126, 13, 2000.)

TABLE 64.4 Design Heights for Daytime Sight Distances

Sight Distance Type	Driver's Eye Height, H_e (ft)	Object Height, H_o (ft)
SSD	3.5	2.0
DSD	3.5	2.0
PSD	3.5	3.5
PVSD	3.5	0
ISD	3.5	3.5

as the object. In Canada, sight distances are measured from a 1.05-m driver's eye height to an appropriate object height for SSD, depending on the prevailing conditions used (ranging from 0 to 0.38 m), a 0.15-m object height for DSD, and a 1.3-m object height for PSD [TAC, 1999]. For PVSD, a 1.05-m driver's eye height (daytime) or a 2-ft vehicle's headlight height (nighttime) and a zero object height have been recommended [Hassan and Easa, 2000].

Simple Highway Curves

Two basic curves are used for connecting straight (tangent) roadway sections in geometric design: a simple circular curve for horizontal alignment and a simple parabolic curve for vertical alignment. Other options include spirals, compound curves, and reverse circular curves for horizontal alignment; and unsymmetrical curves and reverse parabolic curves for vertical alignment. Details on the geometry of these curves can be found in Meyer and Gibson [1980].

Simple Horizontal Curves

A simple **horizontal curve** with radius R and deflection angle I is shown in Fig. 63.5. The basic elements required for laying out a horizontal curve are tangent distance T, external distance E, middle ordinate M, length of chord C, and curve length L. These elements can be easily computed in terms of R and I. For example, the tangent distance T equals R tan (I/2) and the curve length L equals R $I_p/180$. The horizontal curve can also be described by the **degree of curve** (D = 5730/R, where D is in degrees and R is in feet), instead of the radius. However, the degree of curve is no longer used in current geometric design guides.

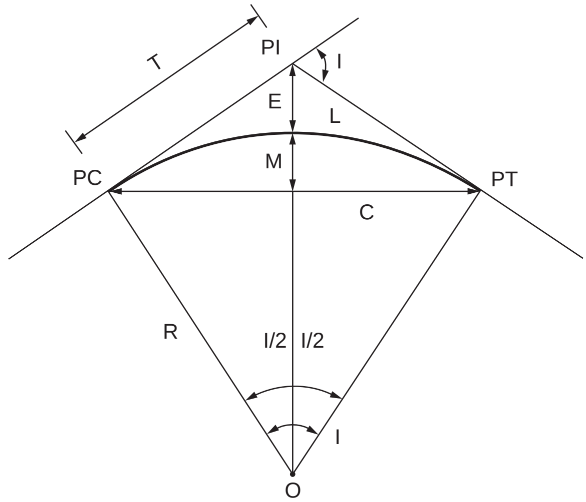


FIGURE 63.5 Geometry of simple horizontal curves.

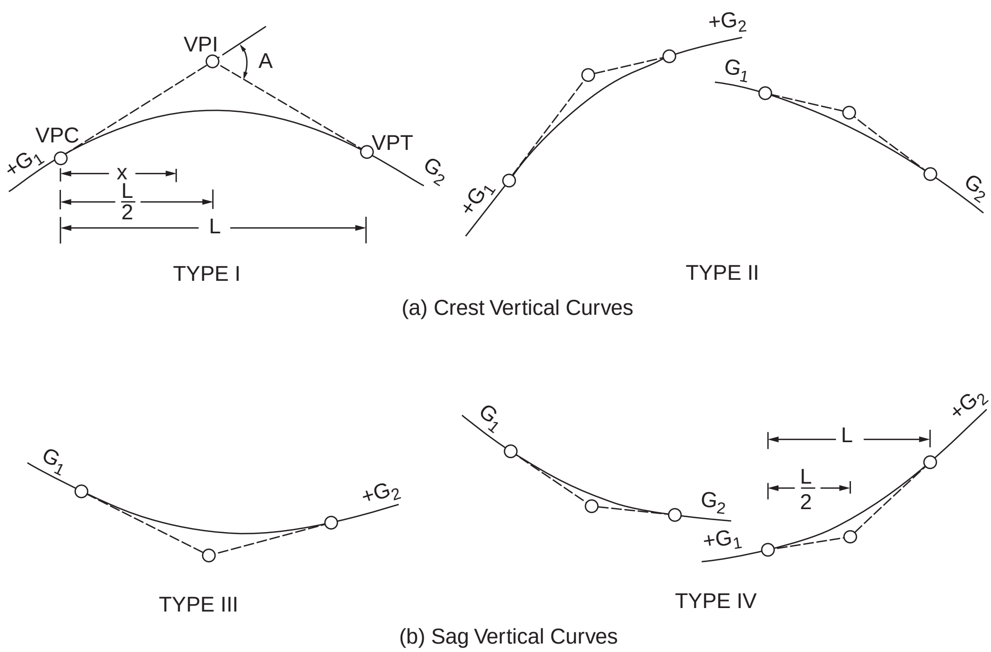


FIGURE 63.6 Types of vertical curves. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

Simple Vertical Curves

Vertical curves are normally parabolic. A simple vertical curve may be a **crest vertical curve** or **sag vertical curve**, as illustrated in Fig. 63.6. In this figure, G_1 and G_2 are the grades of the first and second tangents (in percent), A is the absolute value of the algebraic difference in grades (in percent), L is the curve length measured in a horizontal plane, VPI is the vertical point of intersection, VPC is the vertical point of curvature, and VPT is the vertical point of tangent. For a simple vertical curve, VPI lies in the middle of the curve. The vertical curve parameter, K, is defined as (with L in feet and A in percent)

$$K = \frac{L}{A} \tag{63.4}$$

The elevations of the vertical curve, y, at horizontal distances, x, from VPC are required for laying out the vertical curve. If the elevation of VPC is E_{VPC}, then y is given by

$$y = E_{VPC} + \frac{G_1 x}{100} - \frac{x^2}{200K} \tag{63.5}$$

where y, x, and E_{VPC} are in feet. The location of the highest (lowest) point of the curve, $x_{high}(x_{low})$, is important for pavement drainage requirements. The lowest and highest points exist only for vertical curves of types I and III, respectively (Fig. 63.6). Equating the first derivative of y with respect to x to zero gives

$$x_{high} = KG_1 \tag{63.6}$$

Chapter 7, Section 7.3 presents more details on the layout of horizontal and vertical curves. The radius R of the horizontal curve and the parameter K of the vertical curve are determined based on highway design speed and other parameters, as discussed next.

63.3 Basic Design Applications

The basic elements of geometric design are horizontal alignment, vertical alignment, cross section, and intersection. The design of these elements involves mainly application of the fundamentals discussed in the previous section.

Horizontal Alignment

The horizontal alignment consists of straight roadway sections (tangents) connected by horizontal curves, which are normally circular curves with or without transition (spiral) curves. The basic design features of horizontal alignment include minimum radius, transition curves, **superelevation**, and sight distance. To understand how the minimum radius is determined, the radius–speed relationship is described first.

Radius–Speed Relationship

When a vehicle travels along a horizontal curve, it is forced radially outward by a centrifugal force. The centrifugal force is counterbalanced by the vehicle weight component related to the roadway superelevation and the friction force between the tire and pavement. From the law of mechanics,

$$R = \frac{V^2}{15(0.01e + f)} \tag{63.7}$$

where R = the radius of curve (ft)
V = the vehicle speed (mph)
e = the rate of roadway superelevation (in percent)
f = the side friction (demand) factor

The minimum radius is found based on limiting values of e and f.

Maximum Superelevation

The maximum superelevation, e_{max}, depends on climatic conditions, terrain, location (urban or rural), and frequency of slow-moving vehicles. For open highways, the maximum superelevation is 0.10 or 0.12 in areas without snow and ice; otherwise, the maximum superelevation should be 0.08. A rate of 0.12 may also be used for low-volume gravel roads to facilitate cross drainage. A maximum rate of 0.04 or 0.06 is common in urban areas.

TABLE 63.5 Design Side Friction Factors for Open Highways

Rural Highways, Urban Freeways, and High-Speed Urban Streets		Low-Speed Urban Streets	
Design Speed (mph)	Side Friction Factor, f	Design Speed (mph)	Side Friction Factor, f
20	0.17	20	0.300
30	0.16	25	0.252
40	0.15	30	0.221
50	0.14	35	0.197
60	0.12	40	0.178
70	0.10	45	0.163
80	0.08	—	—

Source: American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.

Maximum Side Friction

Maximum side friction factors for design are established by AASHTO based on field studies. Table 63.5 shows the recommended design values of f for open highways. The higher side friction factors used for low-speed urban streets reflect a tolerable degree of discomfort accepted by drivers and provide a margin of safety compared with actual conditions.

Minimum Radius

For open highways, the minimum radius, R_{min}, for a given design speed is calculated from Eq. (63.7) using the maximum superelevation and maximum side friction factor. When larger radii than R_{min} are used for a given design speed, the required superelevation is found based on a practical distribution of the superelevation rate over the range of curvature. For rural highways, urban freeways, and high-speed urban streets, the recommended design superelevation rates for e_{max} = 0.04 are shown in Table 63.6. Similar tables for other e_{max} values are given by AASHTO. For low-speed urban streets, an accepted procedure is to compute the required superelevation rate with f equal to the maximum value. If the computed value of e is negative, superelevation will not be required (practically, superelevation is set equal to a minimum of 0.015).

For intersection curves and turning roadways, the minimum radii for various design speeds are shown in Table 63.7. The values are based on higher side friction factors and minimum rates of superelevation and are calculated from Eq. (63.7). If conditions allow more than this minimum superelevation, drivers will drive the curve more comfortably because of less friction or will travel at a higher speed. The following two examples illustrate the computations of minimum radius and superelevation rate for high-speed and low-speed urban streets, respectively.

Example 63.2

1. Find the minimum radius on a high-speed urban street with a 50-mph design speed and a 4% maximum superelevation. From Table 63.5, f = 0.14. From Eq. (63.7), the minimum radius is

$$R_{min} = \frac{(50)^2}{15(0.04 + 0.14)} = 926 \text{ ft}$$

   The minimum radius can also be obtained from Table 63.6, for V = 50 mph, as 930 ft.
2. Find the required superelevation rate for a flatter curve on the above street with R = 2000 ft. From Table 63.6, for V = 50 mph, the required superelevation rate is e = 3.2%.

TABLE 63.6 Design Elements Related to Horizontal Curvature (e_{max} = 0.04)

	V_d = 15 mph			V_d = 20 mph			V_d = 25 mph			V_d = 30 mph			V_d = 35 mph			V_d = 40 mph			V_d = 45 mph			V_d = 50 mph			V_d = 55 mph			V_d = 60 mph		
R (ft)	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns	*e* (%)	2 Lns	4 Lns
23000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
20000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
17000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
14000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
12000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
10000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
8000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	51	77	RC	53	80
6000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	48	72	RC	51	77	2.3	61	92
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	44	67	RC	48	72	2.3	59	88	2.6	67	100
4000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	41	62	RC	44	67	2.3	55	83	2.6	66	100	2.8	75	112
3500	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	39	58	RC	41	62	2.2	49	73	2.5	60	90	2.7	69	103	3.0	80	120
3000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	39	58	2.1	43	65	2.4	53	80	2.7	65	97	2.9	74	111	3.3	88	132
2500	NC	0	0	NC	0	0	NC	0	0	RC	38	55	RC	39	58	2.4	50	74	2.5	58	87	2.9	70	104	3.2	82	123	3.5	93	140
2000	NC	0	0	NC	0	0	RC	34	51	RC	38	55	2.3	45	67	2.6	54	81	2.9	64	97	3.2	77	115	3.5	89	134	3.8	101	152
1800	NC	0	0	NC	0	0	RC	34	51	2.1	38	57	2.4	46	70	2.7	56	84	3.0	67	100	3.3	79	119	3.7	94	142	3.9	104	156
1600	NC	0	0	NC	0	0	RC	34	51	2.2	40	60	2.6	50	75	2.9	60	90	3.2	71	107	3.5	84	126	3.8	97	146	4.0	107	160
1400	NC	0	0	NC	0	0	RC	34	51	2.4	44	65	2.7	52	78	3.0	62	93	3.4	76	113	3.7	89	133	3.9	100	149	R_{min} = 1505		
1200	NC	0	0	RC	32	49	2.2	38	57	2.5	45	68	2.9	56	84	3.2	66	99	3.6	80	120	3.9	94	140	4.0	102	153			
1000	NC	0	0	RC	32	49	2.4	41	62	2.7	49	74	3.1	60	90	3.5	72	109	3.8	84	127	4.0	96	144	R_{min} = 1190					
900	NC	0	0	2.1	34	51	2.5	43	64	2.9	53	79	3.2	62	93	3.6	74	112	3.9	87	130	R_{min} = 930								
800	NC	0	0	2.2	35	54	2.6	45	67	3.0	55	82	3.4	66	99	3.8	79	118	4.0	89	133									
700	RC	31	46	2.3	37	56	2.7	46	89	3.2	58	87	3.6	70	105	3.9	81	121	R_{min} = 730											
600	RC	31	46	2.5	41	61	2.9	50	75	3.4	62	93	3.8	74	110	4.0	83	124												
500	2.1	32	48	2.6	42	63	3.1	53	80	3.6	65	98	3.9	75	113	R_{min} = 565														
450	2.2	34	51	2.7	44	66	3.2	55	82	3.7	67	101	4.0	77	116															
400	2.3	35	53	2.9	47	71	3.4	58	87	3.8	69	104	R_{min} = 420																	
350	2.4	37	55	3.0	49	73	3.6	62	93	3.9	71	106																		
300	2.6	40	60	3.2	52	78	3.7	63	95	4.0	73	109																		
250	2.7	42	62	3.4	55	83	3.9	67	100	R_{min} = 300																				
200	3.0	46	69	3.7	60	90	R_{min} = 205																							
150	3.3	51	76	3.9	63	95																								
100	3.8	58	88	R_{min} = 125																										
75	4.0	62	92																											
	R_{min} = 70																													

e_{max} = 4%
R = radius of curve
V_d = assumed design speed
e = rate of superelevation
L = minimum length of runoff (does not include tangent runout) as discussed in "Tangent-to-Curve transition" section
NC = normal crown section
RC = remove adverse crown, superelevate at normal crown slope
Use of e_{max} = 4% should be limited to urban conditions

Source: American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.

TABLE 63.7 Minimum Radii for Intersection Curves and Turning Roadways

Design Speed (mph)	Side Friction Factor, f	Minimum Superelevation, e	Minimum Radius (ft) Computed	Minimum Radius (ft) Rounded for Design
10	0.38	0.00	18	25
15	0.32	0.00	47	50
20	0.27	0.02	92	90
25	0.23	0.04	154	150
30	0.20	0.06	231	230
35	0.18	0.08	314	310
40	0.16	0.09	426	430
45	0.15	0.10	540	540

Note: For design speeds of more than 45 mph, use values for open highway conditions.

Source: American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.

Example 63.3

1. Find the minimum radius for a low-speed urban street with a 40-mph design speed and a 6% maximum superelevation rate. From Table 63.5, f = 0.178, and Eq. (63.7) gives

$$R_{\min} = \frac{(40)^2}{15(0.06 + 0.178)} = 449 \text{ ft } (\text{or } 450 \text{ ft})$$

2. Find the required superelevation rate for a flatter curve on the above street with R = 530 ft. With f equal to the maximum value, Eq. (63.7) becomes

$$530 = \frac{(40)^2}{15(0.01e + 0.178)}$$

from which e = 2.4%.

Horizontal Transition Curves

A horizontal transition (spiral) curve is a curve whose radius continuously changes. It provides a transition between a tangent and a circular curve (simple spiral) or between two circular curves with different radii (segmental spiral). For simple spirals, the radius varies from infinity at the tangent end to the radius of the circular curve at the curve end. For segmental spirals, the radius varies from that of the first circular curve to that of the second circular curve. A transition curve is advantageous because:

1. It provides a natural, smooth path.
2. It provides a length for attaining superelevation.
3. It facilitates pavement widening on curves.
4. It enhances the appearance of the highway.

A practical method for determining the length of a spiral is to use the length required for attaining superelevation.

Method of Attaining Superelevation

The change in cross slope from a section with an adverse crown removed to a fully superelevated section, or vice versa, is achieved over a highway length called the superelevation runoff. The runoff depends on

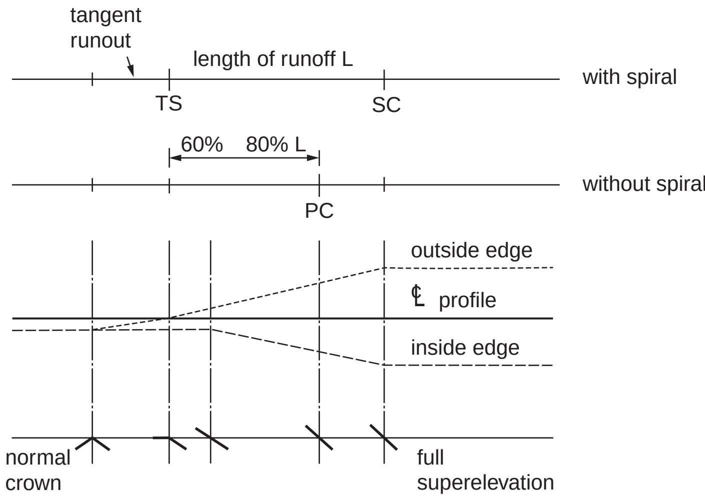


FIGURE 63.7 Attaining superelevation for a curve to the right by rotating pavement about its centerline. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

the design speed, superelevation rate, and pavement width. The minimum length of runoff for two-lane and four-lane highways is obtained from Table 63.6. The superelevation is attained by rotating a crowned pavement about the centerline, the inside edge, or the outside edge. Figure 63.7 shows the method of attaining superelevation for a curve to the right when the pavement is rotated about its centerline. For spiraled circular curves, the length of spiral equals the superelevation runoff. The runoff starts at the tangent-spiral (TS) point and ends at the spiral-curve (SC) point. For unspiraled circular curves, the superelevation runoff is typically positioned such that 60 to 80% of the runoff is on the tangent and the remainder is on the curve. For safety and appearance, angular breaks should be rounded using vertical curves.

Sight Distance on Horizontal Curves

Sight obstacles such as walls, cut slopes, and buildings on the inside of horizontal curves may restrict the available sight distance. Figure 63.8 shows the geometry of lateral clearance and SSD on a four-lane highway. The obstacle lies at the middle of the curve with a lateral clearance measured from the centerline of the inside lane. The required lateral clearance to satisfy a specified sight distance is computed by [Olson et al., 1984]

$$M = R_n\left[1 - \cos\frac{28.65\,S}{R_n}\right], \quad S \le L \tag{63.8}$$

$$M = \frac{L\left(2\,S - L\right)}{8\,R_n}, \quad S > L \tag{63.9}$$

where M = the lateral clearance (ft)
R_n = the radius to the inside-lane centerline (ft)
S = the sight distance, SSD or PSD (ft)
L = the length of the horizontal curve along the inside-lane centerline (ft)

When the obstacle lies near the ends of the curve, the required lateral clearance needs will be less than the values computed by Eqs. (63.8) and (63.9). The required lateral clearance for different obstacle locations have been established [Easa, 1991].

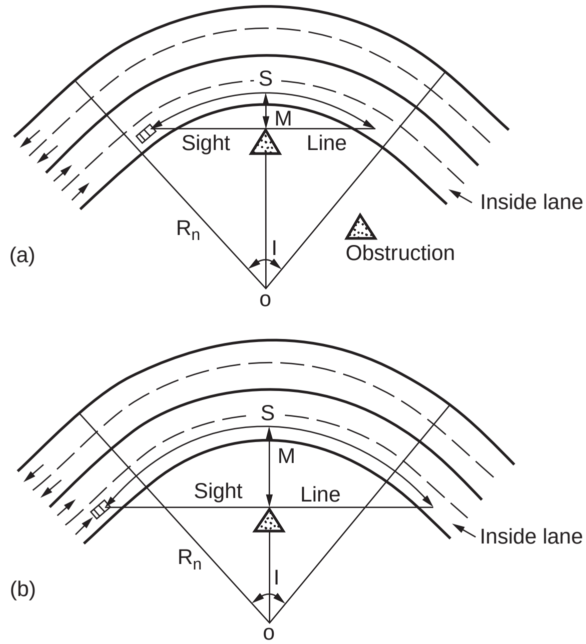


FIGURE 63.8 Lateral clearance on simple horizontal curve with middle obstacle for SSD: (a) $S \le L$ and (b) $S > L$.

Example 63.4

A two-lane highway with 12-ft lanes has a horizontal curve designed for an 800-ft radius, a 50-mph design speed, and I = 40°. Find the required lateral clearance for a middle obstacle to satisfy SSD and PSD of AASHTO. Since the curve radius is typically given for the highway centerline, $R_n = 800 - 6 = 794$ ft. The horizontal curve length is

$$L = \frac{R_n I \pi}{180} = \frac{794 \times 40 \times 3.14}{180} = 554 \text{ ft}$$

From Table 63.2, SSD = 425 ft. Since $S < L$, from Eq. (63.8),

$$M = 794\left[1 - \cos\left(\frac{28.65 \times 425}{794}\right)\right] = 29 \text{ ft}$$

For PSD, from Table 63.2, PSD = 1835 ft. Since $S > L$, from Eq. (63.9),

$$M = \frac{554(2 \times 1835 - 554)}{8 \times 794} = 272 \text{ ft}$$

The required lateral clearance for PSD is clearly impractical because it would exceed the normal **right-of-way** line. Practical design for PSD occurs only for very flat curves.

Vertical Alignment

The vertical alignment consists of straight roadway sections (grades or tangents) connected by vertical curves. The grade line is laid out in the preliminary location study to reduce the amount of earthwork and to satisfy other constraints such as minimum and maximum grades. The basic design features of

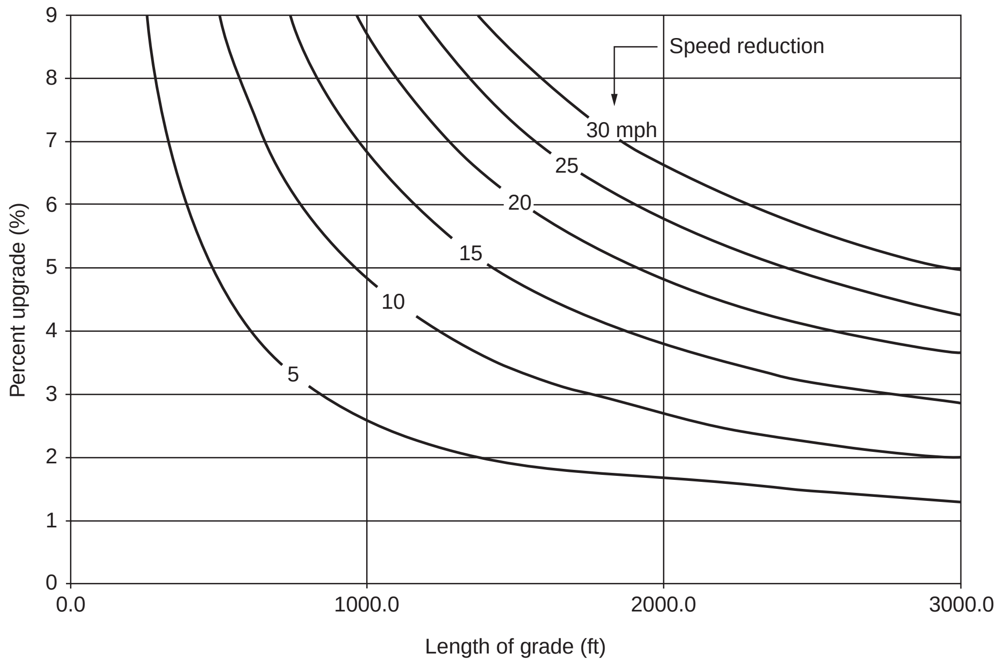


FIGURE 63.9 Critical lengths of grade for design, assumed typical heavy truck of 200 lb/hp, entering speed = 70 mph. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

vertical alignment include grades, critical length of grade, climbing lanes, emergency escape ramps, and vertical curve length.

Grades

Maximum grades for different types of roads and design speeds have been established by AASHTO. Maximum grades of about 5% are considered appropriate for a 70-mph design speed. For a 30-mph design speed, maximum grades generally range from 7 to 12%, depending on topography. For intermediate design speeds, maximum grades lie between the above extremes. For low-volume rural highways, grades may be 2% steeper. Minimum grades are necessary to facilitate surface drainage. For uncurbed roads, the grade may be 0%, provided ditch grades are adequate. For curbed roads, the minimum grade is 0.3%, but a 0.5% grade should be used if possible. For very flat terrain, a grade as low as 0.2% may be necessary.

Critical Length of Grade

The critical length of grade is the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. A 10-mph reduction is used as a general guide, since the accident rate increases significantly when the truck speed reduction is greater than this value. Figure 63.9 shows the critical length of grade for design for a typical design truck with a weight-to-horsepower ratio of 200 lb/hp and a 70-mph entering speed. For grades longer than the critical length, design adjustments to reduce grades or addition of a climbing lane should be considered.

Climbing Lanes

A climbing lane is an extra lane on the upgrade side of a two-lane highway for use by heavy vehicles whose speeds are significantly reduced on upgrades. Climbing lanes improve traffic operation and safety and are justified when the following three criteria are satisfied: (1) the upgrade traffic flow rate exceeds 200 vehicles per hour, (2) the upgrade truck volume exceeds 20 vehicles per hour, and (3) one of the following conditions exists: a 10-mph or greater speed reduction is expected for a typical heavy truck, level-of-service E or F exists on the grade, or a reduction of two or more levels of service occurs when moving from the approach segment to the grade. A climbing lane normally begins where the speed of the design truck is reduced by 10 mph and ends when the design truck regains a speed equal to that at the start of the climbing lane. Details on the design of climbing lanes, including entrance and exit transition tapers, width, signing, and marking, are presented by AASHTO.

Emergency Escape Ramps

An emergency escape ramp is provided on a long, steep downgrade for use by heavy vehicles losing control because of brake failure (caused by heating or mechanical failure). The ramp allows these vehicles to decelerate and stop away from the main traffic stream. There are four basic types of emergency escape ramps: sandpile, descending grade, horizontal grade, and ascending grade. The rolling resistance on the ramps is supplied by the loose sand or an arresting bed of loose gravel. The ascending grade ramp provides a force of gravity opposite the vehicle movement, and therefore its length can be shorter than the descending and horizontal grade ramps. Each ramp type is applicable to a particular topographic situation. More details on emergency escape ramps can be found in an NCHRP synthesis [Witheford, 1992].

Vertical Curve Length

The length of a vertical parabolic curve, based on Eq. (63.4), is computed by

$$L = AK \tag{63.10}$$

where L = the length of vertical curve (ft)
A = the algebraic difference in grades (in percent)
K = the constant

For crest vertical curves, the constant K depends on the sight distance used for design, height of eye above the roadway surface H_e, and height of object above the roadway surface H_o. For sag vertical curves, the design is generally based on a headlight criterion, and the constant K depends on stopping sight distance, headlight height H (2 ft), and the upward divergence of the light beam from the longitudinal axis of the vehicle α (1°). The design (minimum) K values for crest and sag vertical curves are shown in Table 63.8. These values are computed using the formulas shown in the table, where S equals the sight distance for crest curves and the SSD for sag curves. The heights H_e and H_o are given in Table 63.4. When the K value needed for design is greater than 167 ft, pavement drainage near the highest (lowest) point, given by Eq. (63.6), must be more carefully designed. For a small A, the length computed by Eq. (63.10) may be unrealistically small, and it is common practice to express the minimum curve length (in feet) as three times the design speed in miles per hour. The use of zero-length and minimum-length vertical curves has been evaluated by Wooldridge et al. [1999].

A special case is sight distance through a grade separation, where the structure may cut the line of sight and limit the sight distance. The designer may wish to check the available sight distance at the underpass to ensure that it satisfies the required sight distance. Such a check may be made graphically, but equations available in AASHTO can also be used.

Example 63.5

A section of a four-lane highway with partial access control and a 60-mph design speed lies on a combined horizontal curve (R = 1432.5 ft) and crest vertical curve (L = 800 ft), as shown in Fig. 63.10. The length of the horizontal curve is greater than 800 ft. A retaining wall (5 ft high above the pavement) is required for a planned development near the highway. Determine the adequacy of the design for SSD. To check sight distance on the vertical curve, from Table 63.8, K = 151. For A = 2.5%, the required length of the vertical curve, based on Eq. (63.10), is

$$L\left(\text{minimum}\right) = 2.5 \times 151 = 377.5 \text{ ft}$$

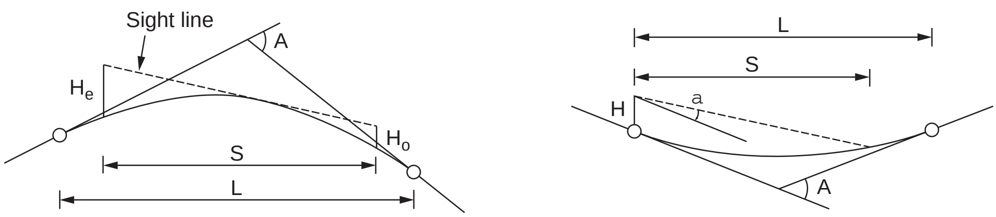
TABLE 63.8 Design Rates of Vertical Curvature K for Crest and Sag Vertical Curves

Design Speed (mph)	K Value for Crest Curves (ft) Stopping Sight Distance	K Value for Crest Curves (ft) Passing Sight Distance	K Value for Sag Curves (ft) Stopping Sight Distance
20	7	180	17
30	19	424	37
40	44	772	64
50	84	1203	96
60	151	1628	136
70	247	2197	181
80	384	2565	231

For S < L:

For crest curves: $$K = \frac{S^2}{200\left(\sqrt{H_e} + \sqrt{H_0}\right)^2}$$

For sag curves: $$K = \frac{S^2}{200\left(H + S \tan \mu\right)}$$



Source: American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, AASHTO, Washington, D.C., 2001. With permission.

Since the vertical curve is 800 ft long, sight distance on the vertical curve is adequate. To check sight distance on the horizontal curve, from Table 63.2, the required SSD is 570 ft. R_n = 1432.5 – 30 – 18 = 1384.5 ft. From Eq. (63.8) for S £ L, the required lateral clearance is 29.2 ft. Since the distance from the retaining wall to the inside-lane centerline, given in the design, is 24 + 6 = 30 ft, the minimum sight distance criterion is also met.

Vertical Transition Curves

Vertical curves are traditionally designed as parabolic curves that are connected directly to the tangents (without transitions). A vertical transition curve has been recently developed for use before and after a parabolic curve [Easa and Hassan, 2000]. The resulting vertical curve, called transitioned vertical curve (Fig. 63.11), consists of transition–parabolic–transition segments. Formulas were developed for the instantaneous elevation, grade, rate of curvature, offset from the first tangent, and the highest (or lowest) point on a transitioned crest (or sag) vertical curve, which is important in drainage design. The minimum length of a transition curve is derived based on the criterion of driver comfort. The transitioned vertical curve exhibits striking similarities to the spiraled horizontal curve, even though the two curves have different mathematical functions. Similar to the spiraled horizontal curve, the transitioned vertical curve is especially useful for sharp vertical alignments.

Cross Section Elements

Typical cross sections for rural highways and urban streets are shown in Fig. 63.12. The cross section elements include the traveled way, shoulders, curbs, medians, sideslopes and backslopes, clear zones,

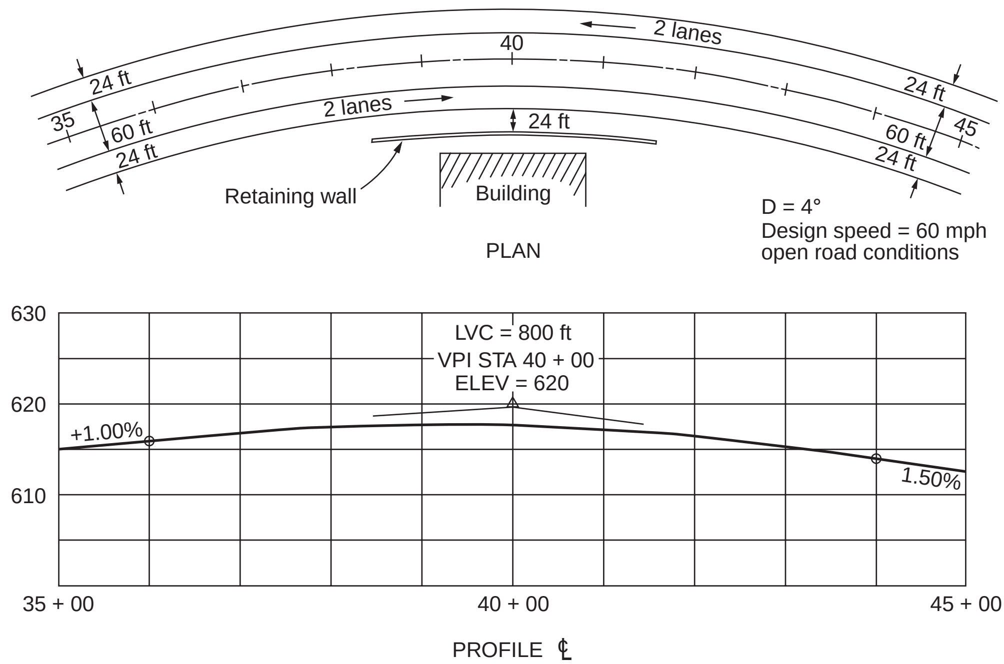


FIGURE 63.10 Example of sight distance analysis for combined horizontal and vertical curves. (From Jack E. Leisch & Associates, *Notes on Fundamentals of Highway Planning and Geometric Design,* Vol. 1, Jack E. Leisch & Associates, Evanston, IL.)

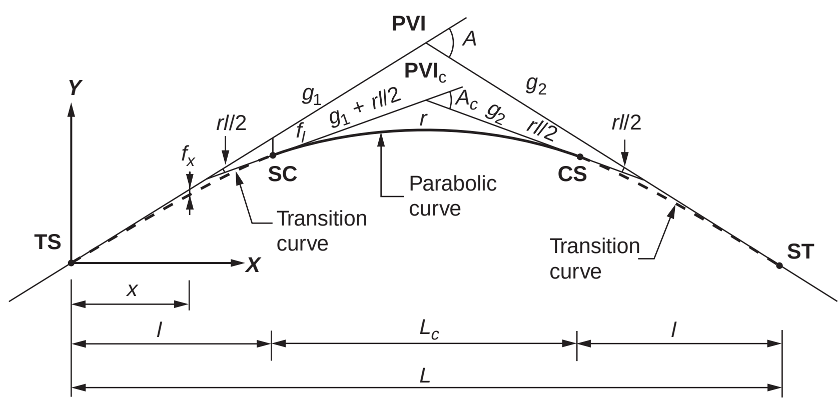


FIGURE 63.11 Vertical transitioned crest curve. (From Easa, S.M. and Hassan, Y., *Transp. Res.*, 34, 481, 2000.)

pedestrian facilities, and bicycle facilities. Higher design guidelines for cross section elements are provided for roads with higher design speeds and volumes.

Traveled Way

The main features of the traveled way are lane width and cross slope. Through-lane width ranges from 10 to 12 ft on most highways, with 12 ft being most common, while auxiliary-lane width ranges from 8 to 12 ft. Auxiliary lanes are used for on-street parking in urban areas; for turning traffic at intersections; for passing slow vehicles on two-lane highways; and for entering, exiting, and weaving traffic on freeways. Safety increases with wider lanes up to a width of 12 ft, but greater lane widths do not offer increased safety. Pavement widening on curves, to provide for vehicle offtracking and additional lateral clearance, is an important design feature. To facilitate cross drainage, all highways are designed with a normal cross

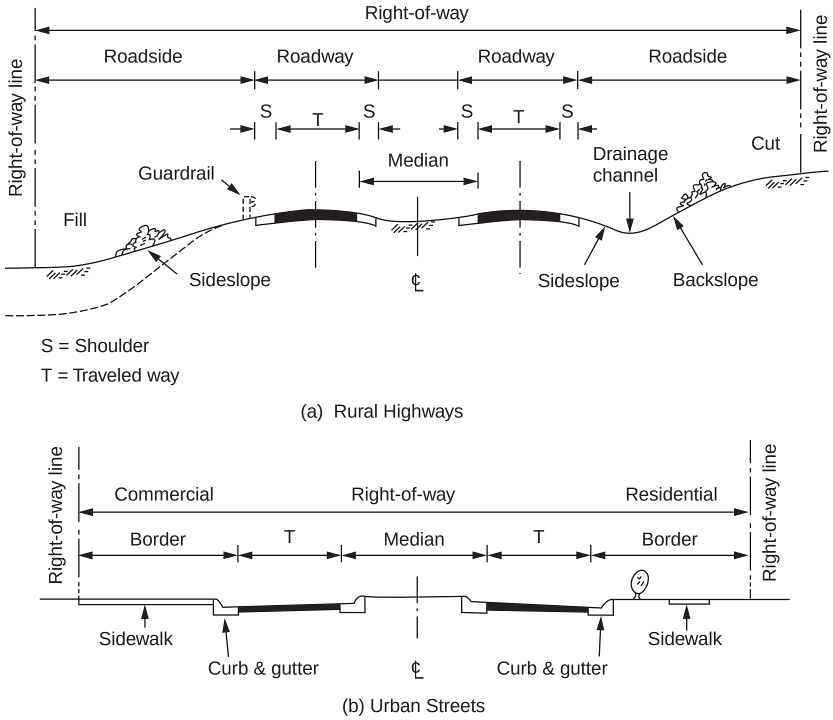


FIGURE 63.12 Typical cross sections for rural highways and urban streets.

slope ranging from 1.5 to 2% for high-type pavement surfaces, 1.5 to 3% for intermediate-type surfaces, and 2 to 6% for low-type surfaces.

Shoulders

Shoulders vary in width from 2 to 12 ft and may be paved or unpaved. Some important advantages of shoulders are providing a space for emergency stopping, maintenance operations, and signs; improving sight distance and highway capacity; and providing a structural support for the pavement. Shoulder cross slopes depend on the type of shoulder construction. Bituminous shoulders should be sloped from 2 to 6%, gravel shoulders from 4 to 6%, and turf shoulders about 8%. The designer should pay attention to the difference between the pavement and shoulder cross slopes on horizontal curves. On the outside of a superelevated section, the difference should not exceed 8%. On the inside, the shoulder and pavement slopes are generally the same. Shoulder contrast is desirable and, when shoulders are used by bicycles, pavement edge lines should be designed as described by the *Manual on Uniform Traffic Control Devices* [FHWA, 1988].

Curbs

Curbs are used for several purposes, including drainage control, pavement edge delineation, esthetics, and delineation of pedestrian walkways. They are used in all types of urban highways, but they are seldom provided on rural highways. The two common types of curbs are barrier curbs and mountable curbs. Barrier curbs have relatively steep faces, designed to prevent vehicles from leaving the roadway. They are used on bridges, piers, and sidewalks, but they should not be used where the design speed exceeds 40 mph, because of the difficulty to retain control of the vehicle after an impact with the curb. Mountable curbs have flat sloping faces, designed to allow vehicles to cross over if required. They are used at median edges, at shoulder inside edges, and to outline channelizing islands at intersections. Barrier and mountable curbs may include a gutter that forms the drainage system for the roadway.

Medians

A median is the portion of a divided highway separating opposing traveled ways. The median width is the distance between the inside-lane edges, including the left shoulders, as shown in Fig. 63.12. The median is primarily provided to separate opposing traffic, but it also provides a recovery area for out-of-control vehicles, a stopping area during emergencies, a storage area for left-turning and U-turning vehicles, and a space for future lanes. Median widths range from a minimum of 4 to 80 ft or more. Wider medians are desirable, but they may lead to inefficient signal operation at intersections. Medians can be depressed, raised, or flushed, depending on width, treatment of median area, and drainage arrangements. A median barrier (a guardrail or a concrete wall) must be considered on high-speed or high-volume highways with narrow medians and on medians with obstacles or a sudden lateral dropoff.

Sideslopes and Backslopes

On fills, sideslopes provide stability for the roadway and serve as a safety feature by being part of a clear zone. In cuts, sideslopes and backslopes form the drainage channels. Sideslopes of 4:1 or flatter are desirable and can be used where the height of a fill or cut is moderate. Sideslopes steeper than 3:1 on high fills generally require roadside barriers. Backslopes should be 3:1 or flatter to facilitate maintenance. Steeper backslopes should be evaluated for soil stability and traffic safety. In rock cuts, backslopes of 1:4 or vertical faces are commonly used. Rounding where slope planes intersect is an important element of safety and appearance.

Clear Zones

The clear zone is the unobstructed, relatively flat area outside the edge of the traveled way, including shoulder and sideslope, for the recovery of errant vehicles. The clear zone width depends on traffic volume, speed, and fill slope. Where a hazard potential exists on the roadside, such as high fill slopes, roadside barriers should be provided. The *Roadside Design Guide* [AASHTO, 1996] provides guidance for design of clear zones, roadside barriers, and sideslopes and backslopes.

Intersections

There are three general types of intersections: at-grade intersections, grade separations without ramps, and interchanges. Selection of a specific intersection type depends on several factors, including highway classification, traffic volume, safety, topography, and highway user benefits. Selection guidelines based on highway classification are given in Table 63.9.

At-Grade Intersections

The objective of intersection design is to reduce the severity of potential conflicts between vehicles, bicycles, and pedestrians. Intersection design is generally affected by traffic factors, physical factors, human factors, and economic factors. Examples of these factors are turning-movement design volumes, sight distance, perception–reaction time, and cost of improvements. There are four basic types of at-grade intersections: three-leg intersections, four-leg intersections, multileg intersections, and roundabouts. The key features of intersection design include capacity analysis, alignment and profile, turning curve radius and width, channelization, median opening, traffic control devices, and sight distance. Details on these design features, along with examples of good designs, are given by AASHTO.

Modern Roundabouts

Modern roundabouts, which have been used less in North America than abroad, differ from the traditional traffic circles or rotaries that have been in use for many years. AASHTO defines two basic principles for modern roundabouts. First, vehicles on the circulatory roadway of the roundabout have the right-of-way, and entering vehicles on the approaches have to wait for a gap in the circulating traffic. Yield signs are used at the entry control. Modern roundabouts are not designed for weaving maneuvers, thus permitting smaller diameters. Second, the centerlines of the entrance roadways intersect at the center of

TABLE 63.9 Selection of Interchanges, Grade Separations, and Intersections Based on Classification

	Consideration Based on Classification	
Intersecting Roads	Rural	Urban
Freeway/freeway	A	A
Freeway/expressway	—	A
Freeway/arterial	B	C
Freeway/collector or local	D	E
Expressway/expressway	—	A
Expressway/arterial	—	F
Expressway/collector or local	—	G
Arterial/arterial	H	H
Arterial/collector or local	I	H or I
Collector or local/collector or local	J	J

Note:

A = Interchange in all cases.

B = Normally interchange, but only grade separation where traffic volume is light.

C = Normally interchange, but only grade separation where interchange spacing is too close.

D = Normally grade separation; alternatively, the collector or local may be used.

E = Normally grade separation, but an interchange may be justified to relieve congestion and serve high-density traffic generators.

F = Normally interchange or intersection; refer to C or G.

G = Normally grade separation, but an intersection may be justified to relieve congestion and serve high-density traffic generators.

H = Normally intersection, but an interchange may be justified where capacity limitation causes serious delay, injury and fatality rates are high, cost is comparable to an intersection, or one arterial may be upgraded to a freeway in the future.

I = Normally intersection; alternatively, the collector or local may be closed.

J = Normally intersection; alternatively, one road may be closed.

Source: Transportation Association of Canada, *Geometric Design Guide for Canadian Roads,* Ottawa, Ontario, 1999. With permission of the Transportation Association of Canada, www.tac-atc.ca.

the roundabout island. Thus, entering traffic is deflected to the right by the central island and by channelization at the entrance.

Rigid objects in the central island, such as monuments, should be avoided because they may pose a safety concern. Instead, esthetic landscaping of the central island and the splitter islands is frequently used. In this case, however, consideration should be given to driver's sight distance needs. There is growing interest in modern roundabouts in the United States, due partially to their success in Europe and Australia. A recent survey showed that the United States has fewer than 50 modern roundabouts, in contrast to 15,000 in France. Current practice and experience with modern roundabouts can be found in a recent NCHRP synthesis [Jacquemart, 1998].

Railroad–Highway Grade Crossings

Horizontal and vertical alignments of a highway approaching a railroad grade crossing require special attention. For horizontal alignment, the highway should intersect the track at a right angle. This layout aids the driver's view of the crossing and reduces conflicting vehicular movements from crossroads and driveways. A right-angle crossing is also preferred for bicyclists because the more the crossing deviates from the ideal 90°, the greater the potential for a bicycle's wheel to be trapped in the flangeway. Crossings should not be located on highway or railroad curves. A roadway curvature may direct a driver's attention toward negotiating the curve instead of the crossing ahead, while a railroad curvature may inhibit the driver's view down the tracks. Crossings with both highway and railroad curves create maintenance problems due to conflicting superelevations.

For vertical alignment, the crossing should be as level as practical to aid sight distance, rideability, braking, and acceleration distances. For crossings without train-activated warning devices, the sight distance should be adequately designed for moving vehicles to safely cross or stop at the railroad crossing and for vehicles to depart from a stopped position at the crossing [AASHTO, 2001].

Intersection Sight Distance

The intersection sight triangle (specified legs along the intersection approaches and across their included corners) must be clear of obstructions that might block the driver's view of potentially conflicting vehicles. The dimensions of the legs of the sight triangle depend on the design speeds of the intersecting roadways and the type of traffic control at the intersection. Two types of sight triangles are considered in intersection design: approach sight triangle and departure sight triangle.

For approach sight triangles, the lengths of the legs should be such that the drivers can see any potentially conflicting vehicles in sufficient time to slow or stop before colliding within the intersection. Typical approach sight triangles to the left and to the right of a vehicle approaching an uncontrolled or yield-controlled intersection are shown in Fig. 63.13A. Note that approach sight triangles are not needed for intersection approaches controlled by stop signs or traffic signals. For departure sight triangles, the clear sight triangle provides a sight distance sufficient for a stopped driver on the minor-road approach to depart from the intersection and enter or cross the major road. Typical departure sight triangles to the left and to the right for a stopped vehicle on the minor road are shown in Fig. 63.13B. Departure sight triangles should also be provided for some signalized intersection approaches. The dimensions of the sight triangles recommended in AASHTO are based on assumptions derived from field observations of driver gap-acceptance behavior.

The surface of the sight triangle is established using a driver's eye height of 3.5 ft and an object height of 3.5 ft above the roadway surface. Clearly, the horizontal and vertical alignments of the intersecting roadways should be considered. An object within the sight triangle will constitute an obstruction if its height is larger than the respective height of the sight triangle. The sight triangle is used to determine required building setbacks, to find whether an existing obstruction should be moved, and to determine appropriate traffic control measures if the obstruction cannot be moved.

AASHTO presented the following cases of intersection sight distance that depend on the type of intersection control: intersection with no control (case A), intersection with stop control on the minor road (case B), intersection with yield control on the minor road (case C), intersection with traffic signal control (case D), intersection with all-way stop control (case E), and left turns from the major road (case F). For cases B and C, consideration is given to whether the vehicle on the minor road is turning left, turning right, or crossing. As an illustration, case B1 (left turn from stop control) is discussed.

Case B1

For this case, the dimensions of the sight triangle (a and b in Fig. 63.13 B) are determined as follows. The length of the sight triangle along the minor road, a, equals the sum of the following distances: (1) distance from the driver's eye to the front of the vehicle (8 ft for passenger cars), (2) distance from the front of the vehicle to the edge of the major-road traveled way (6.5 ft), and (3) distance from the edge of the major-road traveled way to the path of the major-road vehicle (0.5-lane width for vehicles approaching from the left or 1.5 lanes for vehicles approaching from the right).

The length of the sight triangle along the major road, b, is given by

$$ISD = 1.47\, V_{\text{major}}\, t_g \qquad (63.11)$$

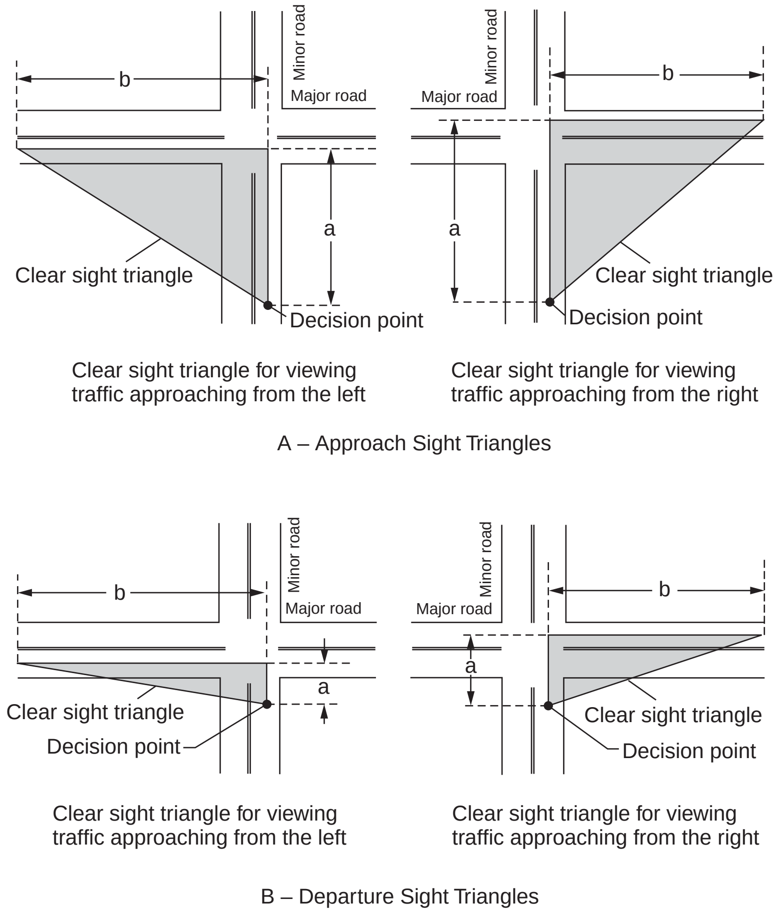


FIGURE 63.13 Intersection sight triangles. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

where ISD = the intersection sight distance (length of the edge of the sight triangle along the major highway (ft))
V_{major} = the design speed of the major road (mph)
t_g = the time gap for the minor-road vehicle to enter the major road (sec)

Based on field observations, t_g equals 7.5 sec for passenger cars, 9.5 sec for single-unit trucks, and 11.5 sec for combination trucks. These values are for a stopped vehicle to turn right or left onto a two-lane highway with no median and grades 3% or less. Adjustments to the gap times for different numbers of lanes of the major road and different approach grades of the minor road are provided by AASHTO. Based on Eq. (63.11), Fig. 63.14 shows the required sight distance along the major road.

The gap-acceptance approach of ISD presented above is based on an NCHRP report by Harwood et al. [1996]. Special cases of intersection sight distance have also been examined. Mason et al. [1989] discussed ISD requirements for large trucks. Gattis [1992] and Gattis and Low [1998] studied sight distance for intersections at other than 90°. Easa [2000] presented a reliability approach to intersection sight distance.

Example 63.6

A four-lane undivided major road with a 60-mph design speed intersects with a minor road controlled by a stop sign. Calculate the lengths of the sight triangle at this intersection for passenger cars turning

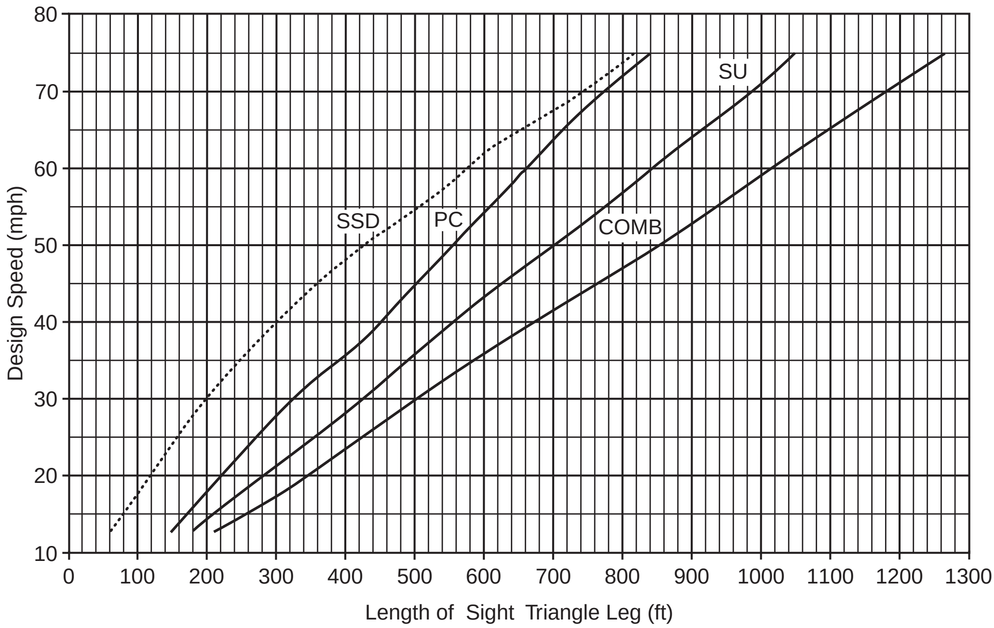


FIGURE 63.14 Intersection sight distance. Case B1: left turn from stop. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

left from the minor road. Assume 12-ft lanes and a 4% upgrade on the minor road. First, compute a = 8 + 6.5 + 18 = 32.5 ft. For passenger cars, $t_g = 7.5$ sec. According to AASHTO, the time gap is increased by 0.5 sec for passenger cars for each additional lane in excess of one to be crossed by the turning vehicle. For a four-lane undivided road, the turning vehicle will need to cross two near lanes, rather than one, thus increasing the time gap from 7.5 to 8.0 sec. Also, the time gap needs to be increased by 0.2 sec for each percent upgrade of the minor road. This brings the final value of t_g to 8.8 sec. Thus, Eq. (63.11) gives

$$ISD = 1.47 ¥ 60 ¥ 8.8 = 776.2 \text{ ft}$$

Suppose that an existing building that cannot be moved restricts the sight distance along the major highway to 660 ft. To make the intersection sight distance adequate, the speed of the major road may be reduced. Substituting for *ISD* = 660 in Eq. (63.11) gives the required speed on the major highway as V_{major} = 51.0 mph, say 50 mph.

Adequate sight distances should also be provided at railroad–highway grade crossings without train-activated warning devices. Two sight distance cases for moving and stopped vehicles are described by AASHTO. Although the 85th-percentile speed of vehicles is normally used in traffic analysis, the 15th-percentile speed should be used for the moving vehicle case [Easa, 1993a].

Interchanges

Interchanges provide the greatest traffic safety and capacity. The basic types of interchanges are shown in Fig. 63.15. The trumpet pattern provides a loop ramp for accommodating the lesser left-turn volume. The three-leg directional pattern is justified when all turning movements are large. The one-quadrant interchange is provided because of topography, even though the volumes are low and do not justify the structure. Simple diamond interchanges are most common for major–minor highway intersections with limited right-of-way. A full-cloverleaf interchange is adaptable to rural areas where the right-of-way is not prohibitive, while a partial-cloverleaf interchange is normally dictated by site conditions and low turning volumes. An all-direction four-leg interchange is most common in urban areas where turning volumes are high.

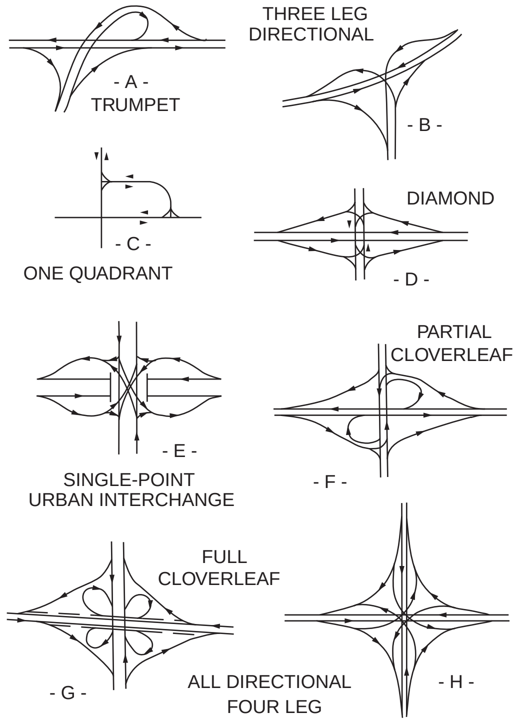


FIGURE 63.15 Interchange configurations. (From AASHTO, *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 2001. With permission.)

In urban areas, where the interchanges are closely spaced, all interchanges should be integrated into a system design that includes the following aspects: (1) interchange spacing, (2) route continuity, (3) uniformity of exit and entrance patterns, (4) signing and marking, and (5) coordination of lane balance and basic number of lanes. Details on these design aspects are presented by AASHTO.

High-occupancy vehicle (HOV) roadways, designated for buses and carpools (typically with three or more persons per vehicle), have been incorporated into urban freeway corridors to improve traffic operations. There are three basic types of HOV lanes. The concurrent-flow type provides an HOV lane normally on the left lane of the roadway in both directions. The HOV lane is separated from the regular-use lanes with a buffer. The contra-flow type designates an HOV lane in the opposing direction of travel. The third type involves physically separating the HOV lane. Details on the design of HOV facilities are given in the *Guide for the Design of High-Occupancy Vehicle Facilities* [AASHTO, 1992a].

Esthetic and General Considerations

Exact adherence to the preceding (specific) design guidelines does not guarantee obtaining a satisfactory and esthetically pleasing design. A number of general guidelines should also be followed in practice for individual horizontal and vertical alignments, combined alignments, and cross sections and intersections.

Individual Alignments

For horizontal alignment, the alignment should be as directional as possible but should conform to the natural contours. The designer should avoid the use of a minimum radius, small deflection angles, sharp curves at the end of long tangents, an abrupt reversal in alignment, and **broken-back curves**. If small deflection angles cannot be avoided, curves should be sufficiently long to avoid the appearance of a kink. For compound curves, the ratio of the flatter radius to the sharper radius should not exceed 1.5.

(a)

(b)

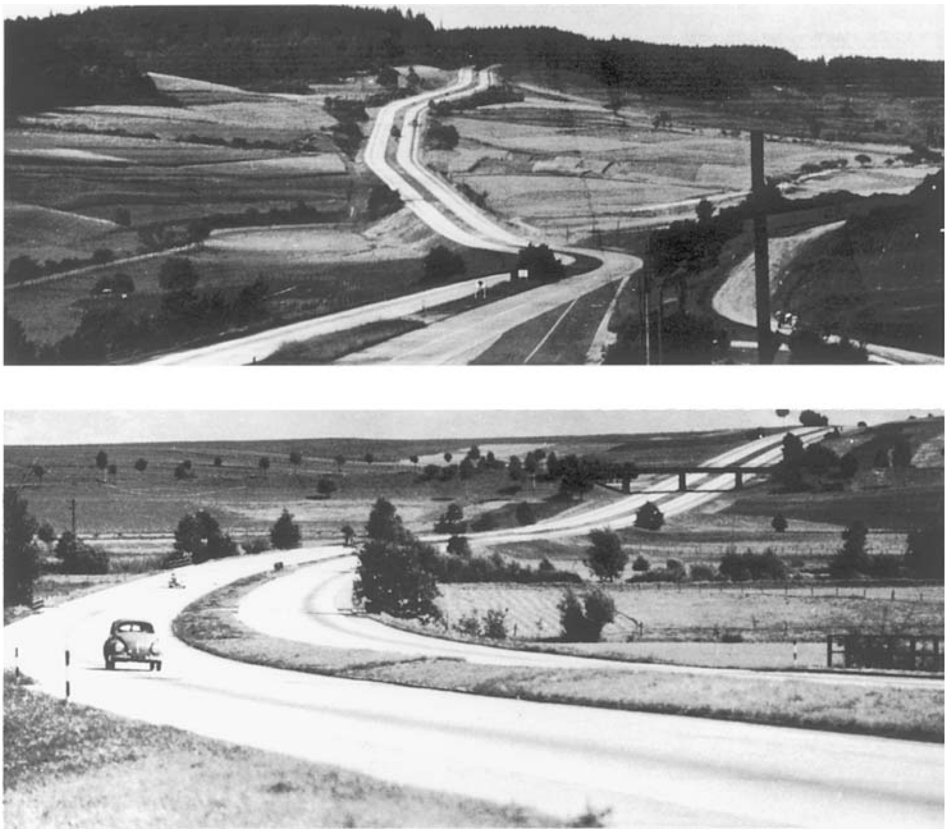


FIGURE 63.16 Example of highways with (a) a long tangent that slashes through the terrain and (b) the more desirable flowing alignment that conforms to the natural contours. (From Transportation Association of Canada, *Geometric Design Guide for Canadian Roads,* Ottawa, Ontario, 1999. With permission of the Transportation Association of Canada, www.tac-atc.ca.)

For vertical alignment, a smooth-flowing profile is preferred to a profile with numerous breaks and short grades. The designer should avoid the hidden-dip type of profiles, broken-back curves, sag vertical curves in cuts, and crest vertical curves on fills. On long grades, the steepest grade should be placed at the bottom, and the grades near the top of the ascent are lightened. Steep grades through important intersections should also be reduced to minimize potential hazards to turning vehicles.

Combined Alignments

Coordination of horizontal and vertical alignments can be achieved during both preliminary location study and final design. A change in horizontal alignment should be made at a sag vertical curve where the driver can readily see the change in direction. However, to avoid distorted appearance, sharp horizontal curves should not be introduced near the low point of the sag vertical curve. If combined horizontal and crest vertical curves cannot be avoided, the horizontal curve should lead the vertical curve. Providing adequate passing opportunities on two-lane highways should supercede other desirable combinations of horizontal and vertical alignments. Alignment coordination is aided by using computer perspectives. Figures 63.16 and 63.17 show two examples of poor and good practice.

Cross Sections and Intersections

Esthetic features of cross section design include well-rounded sideslopes and backslopes, tapered piers and planting below elevated freeways, and spandrel arch bridges where excess vertical clearance is available. On depressed freeways, planting on median barriers, textured retaining walls with luminaire located

FIGURE 63.17 Example of highway perspectives with (a) a short vertical curve imposed on a relatively long horizontal curve, causing the appearance of a settlement of the roadway, and (b) the more desirable alignment where the length of vertical curve is increased to nearly that of the horizontal curve. (From Transportation Association of Canada, *Geometric Design Guide for Canadian Roads,* Ottawa, Ontario, 1999. With permission of the Transportation Association of Canada, www.tac-atc.ca.)

on the top, and landscape planting on the earth slopes aid the roadway appearance. At interchanges, flat earth slopes and long transition grading between cut and fill slopes raise the esthetic level of the area. Landscape development on roadsides and at interchanges is an important design feature for esthetics and safety. Information on the subject is found in *Transportation Landscape and Environmental Design* [AASHTO, 1992b].

Design Aids

A number of design aids are used in geometric design, including physical aids and computer programs. These design aids provide greater flexibility and efficiency during various stages of highway design. Physical aids include templates and physical models. The following types of templates are used in geometric design: horizontal curve, vertical curve, turning vehicle, and sight distance. Horizontal curve templates may be circular curves, three-centered curves, or spiral curves, while vertical curve templates are parabolic. Both types of templates are used in preliminary location study and final design. Turning vehicle templates for design vehicles are used to establish the minimum vehicle path and minimum pavement width for different turning conditions [ITE, 1993].

Sight distance templates are used to measure the available sight distances graphically on highway plans and profiles, especially in a preliminary location study. Each template consists of four parallel horizontal lines. The top line represents the line of sight and the next three lines represent the profile elevations for object height, driver's eye height, and opposing vehicle height, respectively. Physical models are skeletonized, scaled structures easily adjusted to required changes in alignment (design models) or permanent structures used to illustrate the functional plan of a highway or interchange (presentation models).

Computer software is available to carry out all (or specific) aspects of highway geometric design. Available microcomputer programs are regularly published by the Center for Microcomputers in Transportation [McTrans, 1993]. For example, Interactive Computer Assisted Highway Design (ICAHD) is a comprehensive rural highway design program for microcomputers that is used for all geometric design aspects. A sister program, ICAHD Urban Design Model, allows for the design of urban streets and highways. AutoTURN is used for simulating a vehicle turning path in a computer-aided design environment, and SPUI is used for analyzing the geometric design of single-point urban interchanges.

TABLE 63.10 Design Length Requirements for Unsymmetrical Crest and Sag Vertical Curves Based on SSD of AASHTO (Q = 0.4)

Algebraic Difference in Grades (%)	Minimum Length of Crest Vertical Curve (ft)[a]						Length of Sag Vertical Curve (ft)[b]					
	Design Speed (mph)						Design Speed (mph)					
	20	30	40	50	60	70	20	30	40	50	60	70
2	50	70	100	120	330	580	60	90	120	150	180	210
4	50	80	180	470	980	1360	60	160	340	560	820	1130
6	50	170	240	630	1190	1840	140	300	600	970	1410	1920
8	90	250	420	840	1870	2360	220	440	830	1310	1880	2580
10	120	270	530	1210	2230	3230	280	550	1010	1650	2380	3220

[a] Driver's eye height = 3.5 ft; object height = 2.0 ft; a = 1°.
[b] Headlight height = 2 ft.
Source: For minimum length of crest vertical curve: Easa, S.M., Sight distance model for unsymmetrical crest curves, *Transp. Res. Rec.*, 1303, 46, 1991. Note: Values in this reference are based on a 0.5-ft object height and were revised to reflect the new AASHTO 2.0-ft object height. For length of sag vertical curve: Easa, S.M., Sight distance models for unsymmetrical sag curves, *Transp. Res. Rec.*, 1303, 55, 1991.

63.4 Special Design Applications

Special design applications include complex highway curves, three-dimensional alignments, sight distance needs for trucks, and resurfacing, restoration, and rehabilitation projects. Design considerations for these special applications have been recently addressed in the literature and, to a large extent, they supplement the design aspects covered by the AASHTO and TAC guides.

Complex Highway Curves

Complex highway curves consist of two consecutive circular or parabolic curves. Sight distance characteristics of these curves are different from those of simple curves and, consequently, design requirements are different. Four types of complex curves are discussed: unsymmetrical vertical curves, **compound curves**, **reverse curves**, and highway sight-hidden dips (SHDs).

Unsymmetrical Vertical Curves

Unsymmetrical vertical curves may be required on certain occasions because of vertical clearance or other controls. An unsymmetrical vertical curve consists of two consecutive (unequal) parabolic curves with a common tangent at VPI. The curve is described by the parameter Q, which is the ratio of the shorter curve to the total curve length. Design length requirements for unsymmetrical crest and sag vertical curves, based on AASHTO sight distance needs, have been developed. Table 63.10 shows the minimum length requirements for crest and sag vertical curves for $Q = 0.4$. The length requirements for unsymmetrical vertical curves are much greater than those for simple vertical curves.

Recent research has attempted to increase the smoothness and flexibility of unsymmetrical vertical curves. An unsymmetrical vertical curve with two equal arcs has been developed [Easa, 1994a; Easa and Hassan, 1998]. This curve not only is smoother but also improves sight distance, comfort, and esthetics, compared with the traditional unsymmetrical curve. When simple and two-arc unsymmetrical vertical curves cannot satisfy vertical clearance constraints, another type of unsymmetrical vertical curve with three arcs can be used [Easa, 1998].

Compound Horizontal Curves

Compound horizontal curves are advantageous for turning roadways at intersections and interchanges and on open highways, and they are more economical in mountainous terrain. A compound curve consists of two consecutive horizontal curves with different radii turning in the same direction. For obstacles within the sharper (flatter) arc, application of simple curve models, which ignore the flatter (sharper) arc, will underestimate (overestimate) lateral clearance needs. The exact lateral clearance needs for compound curves have been established [Easa, 1993b].

Reverse Curves

Reverse horizontal curves are useful in effecting a change of direction when conditions do not permit the use of simple curves. The alignment reversal reduces the needed lateral clearance on the first curve. Reverse vertical curves (a crest curve followed by a sag curve) are advantageous in hilly and mountainous terrain, and their use is often necessary at interchange ramps. The alignment reversal of the sag curve improves the sight distance and consequently reduces the length requirements of the crest curve. Application of AASHTO guidelines to reverse curves will therefore be conservative. Reverse vertical curves (a crest followed by a sag curve) may produce sight-hidden dips that contribute to passing maneuver accidents on two-lane highways. The AASHTO minimum curvatures for SSD of the crest and sag curves produce unsatisfactory SHDs when the design speed and the algebraic difference in grades of the sag curve are large. An analytical method to identify the locations of SHDs has been developed [Easa, 1994b].

Continuous Two-Dimensional Alignments

Analytical models have been developed to determine the sight distance profile along a continuous two-dimensional highway horizontal or vertical alignment [Hassan et al., 1995; Easa et al., 1996]. The sight distance profile, which shows the available sight distance along the road, can be used to identify locations with inadequate sight distance. The horizontal alignment may consist of any number of tangents, spiral curves, and circular curves. The vertical alignment may consist of any number of tangents, simple parabolic curves, and unsymmetrical curves. The models cover virtually all types of sight obstructions, including single and continuous obstructions in horizontal alignments and crest curves and overpasses in vertical alignments.

Three-Dimensional Alignments

Design Considerations

An analytical model for sight distance analysis on combined horizontal and vertical alignments has been developed by Hassan et al. [1996]. The combined alignment, which is three-dimensional in nature, is represented using the finite-element method (Fig. 63.18). The model was used to examine the required radius of a horizontal curve and the required length of a vertical curve. By comparing the two-dimensional and three-dimensional results, it was found that current two-dimensional practice might significantly underestimate or overestimate the design requirements. More details on these and other developments, such as headlight sight distance for three-dimensional alignments, can be found in a number of papers cited in Hassan et al. [1998].

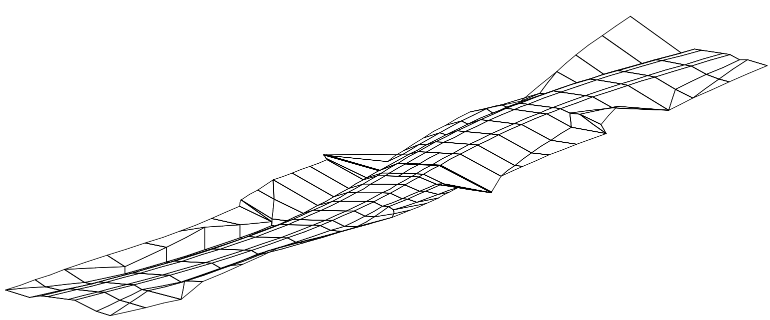


FIGURE 63.18 Finite-element modeling of highway alignment for sight distance analysis. (From Hassan, Y. et al., *Transp. Res. Rec.,* 1523, 1, 1996.)

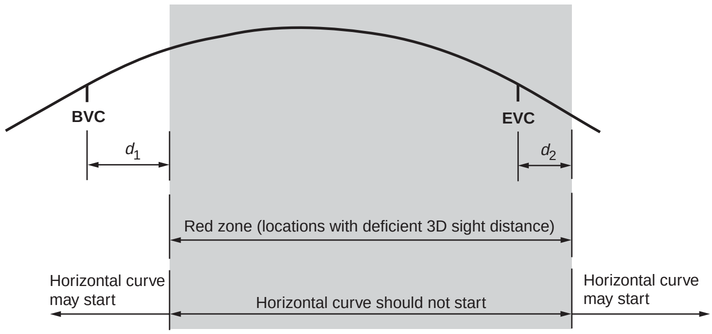


FIGURE 63.19 Concept of red zones on crest vertical curves. (From Hassan, Y. and Easa, S.M., *J. Transp. Eng. ASCE*, 124, 343, 1998.)

Red Zones

The analysis of "red zones" is intended to identify the locations on a vertical curve where a combined horizontal curve should not start [Hassan and Easa, 1998]. It is an alternative to the three-dimensional design that provides a quantitative representation of the AASHTO general guidelines for combined horizontal and vertical curves. Based on sight distance needs, two types of red zones can develop on an alignment designed using current two-dimensional practice: PVSD red zones and SSD red zones. Using analytical three-dimensional models, the range of red zones where a horizontal curve should not start relative to a crest or a sag vertical curve was determined (Fig. 63.19). The analysis of red zones can help designers avoid inadequate coordination of horizontal and vertical alignments on new highways and evaluate the adequacy of existing combined alignments.

RRR Projects

Resurfacing, restoration, and rehabilitation projects, such as minor widening of lanes, can improve highway safety by upgrading selected features of existing highways without the cost of full reconstruction. The AASHTO policy is intended for new highway and major reconstruction projects, but it is not for RRR projects. To address the need for design guidelines for these projects, a study by the Transportation Research Board examined the safety cost-effectiveness of geometric design guidelines and recommended several design practices rather than the minimum guidelines [TRB, 1987]. The recommendations fall into five categories: safety-conscious design process, design practices for key highway features, other design procedures and assumptions, planning and programming RRR projects, and safety research and training. Subsequent research on safety effectiveness of highway design features has been conducted and reported in six volumes [Zegeer and Council, 1992].

63.5 Emerging Design Concepts

Design Consistency

While traditional geometric design deals with individual highway elements, design consistency explicitly addresses the contiguous combination of basic design elements or the longitudinal variations of such features as horizontal alignment, vertical alignment, and cross section [TAC, 1999]. Design consistency includes three main areas: operating speed consistency, safety consistency, and driver performance consistency [Gibreel et al., 1999]. The most recent work on design consistency was a major 1999 FHWA study that expanded on an earlier FHWA study [Krammes et al., 1995] in two directions: to expand the speed-profile model and to investigate three promising design consistency rating methods [Fitzpatrick et al., 2000a, 2000b]. Operating speed models for three-dimensional alignments have been developed [Gibreel et al., 2001].

Design Flexibility

Design flexibility is a design exception process through which departures from design guidelines are addressed. Design flexibility is more frequently applied to urban and rural nonfreeways that are typically responsive to site-specific requirements, where each problem area is addressed within its context and constraints (e.g., roads that are adjacent to historic properties). FHWA has published the *Flexibility in Highway Design* guide, which encourages governing agencies to evaluate design elements that may adhere to safety guidelines while complementing the unique character of the surrounding community [FHWA, 1997]. To achieve this balanced design perspective, the guide suggested seven possible methods for the transportation professional. The wider acceptance of design flexibility in Europe has been shaped by the commitment to create a roadway environment that addresses safety, capacity, economic, and environmental concerns [FHWA, 2001].

Safety Audits

A road safety audit, a concept first developed in Australia, New Zealand, and the United Kingdom, was introduced in North America in 1997. A road safety audit is a tool that can be used by safety professionals to proactively ensure that road safety is explicitly considered and adequately addressed before a road is planned, designed, and built [TAC, 1999]. A road safety audit is defined as a formal examination of the safety performance of an existing or proposed project by an independent, qualified examiner. The objectives of a road safety audit are to identify the potential for road safety problems for all system users and to ensure that all measures for reducing safety problems are adequately considered. With these objectives, several beneficial outcomes are expected: reduction in collision frequency and severity, greater prominence of road safety concerns in the minds of road designers, and reduction in the need for subsequent rehabilitation work to “fix” the problems.

Human Perception

As previously mentioned, driver characteristics affect road design and operations. Traditionally, human factors are used in the design of highway signs, markings, traffic signals, and sight distance. Research work in progress, however, has taken human factor research one step further by examining the effect of vertical alignment on the driver’s perception of horizontal curves. Interest is also emerging in the development of designs in accordance with prevalent driver expectancies based on specific criteria. Such criteria can be integrated into safety-conscious (proactive) road planning that ensures that road safety is explicitly addressed in the planning stage, thus reducing the subsequent need for reactive mitigation measures [Sayed and Easa, 2000].

Smart Design

The use of smart technologies in geometric design and road safety, such as driving simulators and black boxes, is growing. A driving simulator is a personal computer-based interactive tool that includes a vehicle dynamics model, visual and auditory feedback, steering “feel” feedback, and a driver performance system. The driving scene is controlled by events that include roadways, intersections, and traffic control devices. Driving simulators offer complete control of environmental factors and can explore a wide range of issues related to driver performance and road design. Projects can range from perception of three-dimensional alignments to intelligent transportation system experiments. A black box (a small computer hidden in a vehicle) stores data about vehicle speed, air bag, and seat belt at the time of a collision. Some new models also capture 5 sec of data before the impact, such as vehicle speed and gas pedal position. The data can be downloaded from the black box after a collision and used by vehicle and highway engineers. These devices could revolutionize some aspects of collision research.

63.6 Economic Evaluation

The objective of economic evaluation is to evaluate and rank alternative highway improvements so that a selection can be made. Economic evaluation of major improvements requires information about the highway costs and benefits. The costs consist of capital costs and maintenance and operating costs. Capital costs include engineering design, right-of-way, and construction. Maintenance and operating costs include roadside maintenance, snow removal, and lighting and are incurred annually over the service life of the facility. The benefits are normally based on highway user benefits that are the savings in the costs of vehicle operation, travel time, and accidents. For local highway safety improvements, such as improving sight distance, the benefits normally are based on reduced accident costs.

To carry out an economic evaluation, the change in the value of money over the life of the facility must be considered. All costs and benefits of each alternative are combined into a single number (measure) using economic evaluation methods. These include net present value, benefit–cost ratio, equivalent uniform annual cost, and internal rate of return. It is stressed that the results of economic evaluation must be set alongside other strategic and nonmonetary considerations before the policy maker can reach a final decision. Further details on full project economic evaluation, along with example calculations, are presented in *A Manual on User Benefit Analysis of Highway and Bus Transit Improvements* [AASHTO, 1977].

A simplified method for estimating user cost savings for highway improvements has been developed [TAC, 1993]. Simple "look-up" tables are presented for the following facility types: (1) two-lane highway, (2) two-lane highway with passing lane, (3) four-lane divided arterial highway, (4) signalized highway intersection, and (5) highway interchange. For each facility type, the tables give estimates of road user costs over a range of traffic levels. The tables are not intended to replace full project economic evaluation, but they provide a low-cost initial screening at the early stages of planning and designing highway improvements.

63.7 Summary: Key Ingredients

The fundamentals of highway geometric design and their applications are presented in this chapter. They include highway type, design controls, sight distance, and simple highway curves that influence the design of four basic highway elements: horizontal alignments, vertical alignments, cross sections, and intersections. Recent information on the design of complex highway curves, three-dimensional alignment design, sight distance needs for trucks, design considerations for RRR projects, and economic evaluation is presented. Emerging design concepts, including design consistency, design flexibility, safety audits, human perception, and smart design, are described.

Geometric design guidelines promote safety, efficiency, and comfort for the road users. However, strict application of these guidelines will not guarantee obtaining a good design. The following key ingredients are also required:

1. Consistency: Geometric design should provide positive guidance to the drivers to achieve safety and efficiency and should avoid abrupt changes in guidelines. Highways must be designed to conform to driver expectations.
2. Esthetics: Visual quality can be achieved by careful attention to coordinating horizontal and vertical alignments and to landscape developments. The process can be greatly aided by using computer perspectives and physical models.
3. Engineering judgment: Experience and skills of the designer are important in producing a good design. Considerable creativity is required in developing a design that addresses environmental and economic concerns.

Future research in geometric design will likely involve a number of areas, such as human factors, smart technologies, design consistency, design flexibility, and reliability analysis. In particular, the link between geometric design and human factors (which contribute to 90% of road collisions) will require a significant research effort to improve our understanding of the close link between how roads are built and how people use them. The dynamic nature of geometric design will aid these developments.

Defining Terms

30th highest hour volume — The hourly volume that is exceeded by 29 hourly volumes during a designated year.

Average daily traffic (ADT) — The total traffic volume during a given time period (in whole days greater than 1 day and less than 1 year) divided by the number of days in that time period.

Average running speed — The distance divided by the average running time to traverse a segment of highway (time during which the vehicle is in motion).

Broken-back curve — An alignment in which a short tangent separates two horizontal or vertical curves turning in the same direction.

Compound curve — A curve composed of two consecutive horizontal curves turning in the same direction.

Crest vertical curve — A curve in the longitudinal profile of a road having a convex shape.

Degree of curve — The central angle subtended by a 100-ft arc.

Horizontal curve — A curve in plan that provides a change of direction.

Level of service — A qualitative measure that describes operating conditions of a traffic stream and their perception by motorists and passengers.

Operating speed — The speed at which drivers are observed operating their vehicles during free-flow conditions (normally the 85th percentile of the distribution of observed speeds).

Peak-hour factor — The ratio of the total hourly volume to the maximum flow rate during a 15-min period (largest number of vehicles during a 15-min period multiplied by 4).

Reverse curve — A curve composed of two consecutive horizontal or vertical curves turning in opposite directions.

Right-of-way — The land area (width) acquired for the provision of a highway.

Sag vertical curve — A curve in the longitudinal profile of a road having a concave shape.

Superelevation — The gradient across the roadway on a horizontal curve measured at right angles to the center line from the inside to the outside edge.

Turning roadway — A separate roadway to accommodate turning traffic at intersections or interchanges.

References

Alexander, G. and Lunenfeld, H., *A User's Guide to Positive Guidance,* U.S. Department of Transportation, Washington, D.C., 1990.

American Association of State Highway and Transportation Officials, *A Manual on User Benefit Analysis of Highway and Bus Transit Improvements*, AASHTO, Washington, D.C., 1977.

American Association of State Highway and Transportation Officials, *Guide for the Design of High-Occupancy Vehicle Facilities*, AASHTO, Washington, D.C., 1992a.

American Association of State Highway and Transportation Officials, *Transportation Landscape and Environmental Design*, AASHTO, Washington, D.C., 1992b.

American Association of State Highway and Transportation Officials, *Roadside Design Guide*, AASHTO, Washington, D.C., 1996.

American Association of State Highway and Transportation Officials, *Guide for the Development of New Bicycle Facilities*, AASHTO, Washington, D.C., 1999.

American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, AASHTO, Washington, D.C., 2001.

American Association of State Highway and Transportation Officials, *Guide for the Planning, Design, and Operation of Pedestrian Facilities,* AASHTO, Washington, D.C., 2002 (forthcoming).

Easa, S.M., Lateral clearance to vision obstacles on horizontal curves, *Transp. Res. Rec.*, 1303, 22, 1991.

Easa, S.M., Should vehicle 15-percentile speed be used in railroad grade crossing design? *ITE J.,* 63, 37, 1993a.
Easa, S.M., Lateral clearance needs on compound horizontal curves, *J. Transp. Eng. ASCE*, 119, 111, 1993b.
Easa, S.M., New and improved unsymmetrical vertical curve for highways, *Transp. Res. Rec.,* 1445, 95, 1994a.
Easa, S.M., Design considerations for highway sight-hidden dips, *Transp. Res. B*, 28, 17, 1994b.
Easa, S.M., Three-arc vertical curve for constrained highway alignments, *J. Transp. Eng. ASCE*, 124, 162, 1998.
Easa, S.M., Reliability approach to intersection sight distance, *Transp. Res. Rec.,* 1701, 42, 2000.
Easa, S.M. and Hassan, Y., Design requirements of equal-arc unsymmetrical curves, *J. Transp. Eng. ASCE*, 124, 404, 1998.
Easa, S.M. and Hassan, Y., Transitioned vertical curves: I. Properties, *Transp. Res.*, 34, 481, 2000.
Easa, S.M., Abd El Halim, A., and Hassan, Y., Sight distance evaluation on complex highway vertical alignments, *Can. J. Civ. Eng*., 23, 577, 1996.
Federal Highway Administration, *Manual on Uniform Traffic Control Devices,* U.S. Department of Transportation, Washington, D.C., 1988.
Federal Highway Administration, *Highway Functional Classifications: Concepts, Criteria, and Procedures,* U.S. Department of Transportation, Washington, D.C., 1989.
Federal Highway Administration, *Flexibility in Highway Design,* U.S. Department of Transportation, Washington, D.C., 1997.
Federal Highway Administration, *Geometric Design Practices for European Roads,* U.S. Department of Transportation, Washington, D.C., 2001.
Fitzpatrick, K. et al., Speed Prediction for Two-Lane Rural Highways, Report FHWA-RD-9-171, Federal Highway Administration, Washington, D.C., 2000a.
Fitzpatrick, K. et al., Alternative Design Consistency Rating Methods for Two-Lane Rural Highways, Report FHWA-RD-9-172, Federal Highway Administration, Washington, D.C., 2000b.
Garber, N.J. and Hoel, L.A., *Traffic and Highway Engineering,* West Publishing Company, New York, 2001.
Gattis, J.L., Effects of design criteria on local streets sight distance, *Transp. Res. Rec.,* 1303, 33, 1991.
Gattis, J.L., Sight distance design for curved roadways with tangential intersections, *Transp. Res. Rec.,* 1356, 20, 1992.
Gattis, J.L. and Duncan, J., Geometric design for adequate operational preview of road ahead, *Transp. Res. Rec.,* 1500, 139, 1995.
Gattis, J.L. and Low, S.T., Intersection angle geometry and the driver's field of view, *Transp. Res. Rec.,* 1612, 10, 1998.
Gibreel, G., Easa, S.M., and El-Dimeery, I., Prediction of operating speed on three-dimensional highway alignments, *J. Transp. Eng. ASCE*, 127, 20, 2001.
Gibreel, G. et al., State of the art of geometric design consistency, *J. Transp. Eng. ASCE*, 125, 305, 1999.
Harwood, D.W. et al., Intersection Sight Distance, NCHRP Report 383, National Research Council, Washington, D.C., 1996.
Hassan, Y. and Easa, S.M., Design considerations of sight distance red zones on crest vertical curves, *J. Transp. Eng. ASCE*, 124, 343, 1998.
Hassan, Y. and Easa, S.M., Modeling of required preview sight distance, *J. Transp. Eng. ASCE*, 126, 13, 2000.
Hassan, Y., Easa, S.M., and Abd El Halim, A., Sight distance on horizontal alignments with continuous lateral obstructions, *Transp. Res. Rec.,* 1500, 31, 1995.
Hassan, Y., Easa, S.M., and Abd El Halim, A., Analytical model for sight distance analysis on three-dimensional highway alignments, *Transp. Res. Rec.,* 1523, 1, 1996.
Hassan, Y., Easa, S.M., and Abd El Halim, A., Highway alignments: three-dimensional problem and three-dimensional solution, *Transp. Res. Rec.,* 1612, 17, 1998.
Institute of Transportation Engineers, Turning Vehicle Templates: Metric System, Publication LP-022A, ITE, Washington, D.C., 1993.

Jacquemart, G., Modern Roundabout Practice in the United States, NCHRP Synthesis 264, National Research Council, Washington, D.C., 1998.
Koepke, F. and Levinson, H., Access Management Guidelines for Activity Centers, NCHRP Report 348, National Research Council, Washington, D.C., 1992.
Krammes, R.A. et al., Horizontal Alignment Design Consistency for Rural Two-Lane Highways, Report FHWA-RD-94-034, Federal Highway Administration, McLean, VA, 1995.
Mason, J.M., Jr., Fitzpatrick, K., and Harwood, D.W., Intersection sight distance requirements for large trucks, *Transp. Res. Rec.,* 1208, 47, 1989.
McTrans — Center for Microcomputers in Transportation, *Software and Source Book,* Transportation Research Center, University of Florida, Gainesville, FL, 1993.
Meyer, C.F. and Gibson, D.W., *Route Surveying and Design*, International Textbook Co., Scranton, PA, 1980.
Olson, P.L. et al., Parameters Affecting Stopping Sight Distance, NCHRP Report 270, National Research Council, Washington, D.C., 1984.
Pietrucha, M.T. and Opiela, K.S., Safe accommodation of pedestrians at intersections, *Transp. Res. Rec.*, 1385, 12, 1993.
Sayed, T. and Easa, S.M., Need for unified multidisciplinary approach to road safety: I. Road engineering and human factors, *Can. Civ. Eng.*, 17, 6, 2000.
Transportation Association of Canada (TAC), *Highway User Cost Tables: A Simplified Method of Estimating User Cost Savings for Highway Improvements*, TAC, Ottawa, Ontario, 1993.
Transportation Association of Canada (TAC), *Geometric Design Guide for Canadian Roads*, TAC, Ottawa, Ontario, 1999.
Transportation Research Board, Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation, Special Report 214, National Research Council, Washington, D.C., 1987.
Transportation Research Board, Highway Capacity Manual, Special Report 209, National Research Council, Washington, D.C., 2000.
Witheford, D., Truck Escape Ramps, NCHRP Synthesis of Highway Practice 178, National Research Council, Washington, D.C., 1992.
Wooldridge, M.D., Nowlin, R.L., and Parham, A.H., Evaluation of zero-length vertical curves, *Transp. Res. Rec.*, 1658, 52, 1999.
Zegeer, C.V. and Council, F.F., *Safety Effectiveness of Highway Design Features, Volume III: Cross Sections,* Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 1992.

Further Information

American geometric design guidelines for urban and rural highways are found in *A Policy on Geometric Design of Highways and Streets* (2001) by AASHTO. Canadian design guidelines are found in *Geometric Design Guide for Canadian Roads* (1999) by TAC. These references contain design practices in universal use.

Geometric Design Projects for Highways: An Introduction (2000), by J.G. Schoon, published by the American Society of Civil Engineers, is particularly helpful for understanding the preliminary location study of a proposed highway. The book illustrates the design procedures with detailed case studies.

Recent Geometric Design Research for Improved Safety and Operations (2001), NCHRP Synthesis 299, by Kay Fitzpatrick and Mark Wooldridge, published by the Transportation Research Board, is an excellent review of the geometric design research published during the 1990s, particularly research with improved safety and operations implications. The review is presented in agreement with the primary sections of the AASHTO Green Book. The synthesis will be of interest to roadway geometric design, safety, and operations engineers, researchers, and managers.

Human Factors for Highway Engineers (2002), edited by R. Fuller and J.A. Santos, published by Pergamon Press, provides psychological knowledge and insight to help match roadway and transportation system design to human strength, limitations, and variability in performance; an understanding of human contributory factors in collisions; and the undertaking of informed safety audits and reviews.

Available microcomputer programs for geometric design are described in *Software and Source Book*, published annually by the Center for Microcomputers in Transportation. For subscription information, write McTrans, Transportation Research Center, University of Florida, Gainesville, FL 32611–2083; phone (904) 392–0378; or fax (904) 392–3224.