

CHAPTER 2

DESIGN ELEMENTS AND DESIGN CONTROLS

2.0 INTRODUCTION

There are many factors that contribute to the decisions required for the geometric design elements and controls utilized in the location and the design of the various types of highways. Without some type of basic framework of design controls, the judgment of the individual designers may vary considerably. This Chapter presents the guidelines required to tailor the highway to the terrain, to the controls of the land usage and to the type of traffic anticipated. In applying these guidelines, it is important to follow the basic principle that consistency in design is of major importance on any section of highway.

Additional sources of information and criteria to supplement the design elements and related concepts presented in this Chapter are contained in the 2004 AASHTO Green Book, the *HCM* and the *MUTCD*.

2.1 HORIZONTAL ALIGNMENT

To obtain balance in highway design, all geometric elements should be designed to provide safe, continuous operation, as far as economically practical, through the use of design speed as an overall design control. Where curvature in the highway alignment is required, it should be based on an appropriate relationship between design speed and curvature and their joint relationships with superelevation and side friction. These factors shall be properly balanced to produce an alignment that is safe, in harmony with the topography and adequate for the design classification of the roadway or highway. These factors are discussed at length in subsequent Sections. To avoid poor design practices, the following general controls for horizontal alignment should be used:

1. Alignment should be as directional as practical but should be consistent with the topography and with preserving developed properties and community values.
2. In alignment developed for a given design speed, the minimum radius of curvature for that speed should be avoided wherever practical.
3. Consistent alignment should always be sought.
4. For small deflection angles, curves should be sufficiently long enough to avoid the appearance of a kink.
5. Avoid sharp curvature on long, high fills.
6. Caution should be exercised in the use of compound circular curves.
7. Abrupt reversals in alignment should be avoided.
8. The "broken-back" or "flat-back" arrangement of curves (with a short tangent between two curves in the same direction) should be avoided except where very unusual topographical or right-of-way conditions make other alternatives impractical.
9. To avoid the appearance of inconsistent distortion, the horizontal alignment should be coordinated carefully with the profile design as presented in [Section 2.3](#). Such coordination is especially important at railroad-highway grade crossings.

For additional information concerning general and design considerations for horizontal alignment and additional presentations on the practical application of the relevant criteria, refer to the section "General Controls for Horizontal Alignment" in the 2004 AASHTO Green Book, Chapter 3.

2.2 VERTICAL ALIGNMENT

As with other design elements, the characteristics of vertical alignment are influenced greatly by basic controls related to design speed, highway functional classifications and the terrain conditions. Within these basic controls, there are several general controls for vertical alignment that should be considered that include:

1. A smooth gradeline with gradual changes, as consistent with the type of highways, roads or streets and the character of terrain, should be sought for in preference to a line with numerous breaks and short lengths of grades.
2. The "roller-coaster" or the "hidden-dip" type of profile should be avoided.
3. Undulating gradelines involving substantial lengths of momentum grades should be evaluated for their effect on traffic operation.
4. A broken-back gradeline (two vertical curves in the same direction separated by short sections of tangent grade) generally should be avoided, particularly in sags where the full view of both vertical curves is not pleasing.
5. It may be preferable, on long grades, to place the steepest grades at the bottom and flatten the grades near the top of the ascent or to break the sustained grade by short intervals of flatter grade.
6. Where at-grade intersections or railroad-highway grade crossings occur on roadway sections with moderate to steep grades, it is desirable to reduce the grade through the intersection or railroad-highway grade crossing.
7. Sag vertical curves should be avoided in cuts unless adequate drainage can be provided.

For additional information concerning general and design considerations for vertical alignment and additional presentations on the practical application of the relevant criteria, refer to the section "General Controls for Vertical Alignment" in the 2004 AASHTO Green Book, Chapter 3.

2.3 CONTROLS FOR COMBINATION HORIZONTAL AND VERTICAL ALIGNMENTS

Horizontal and vertical alignments represent permanent design elements which warrant thorough examination and study. They should not be designed independently, but should complement each other to avoid alignment deficiencies. Excellence in the design of each and the integration of their interrelated concepts results in a completed highway that provides increased safety, usefulness, uniform speeds and improved appearances on which to travel.

The proper combination of horizontal and vertical alignment is obtained through engineering studies with consideration given to the following general guidelines:

1. Curvature and grades should be in proper balance. Tangent alignment or flat curvature at the expense of steep or long grades and excessive curvature with flat grades both represent poor design. A logical design that offers the best combination of safety, capacity, ease and uniformity of operation and pleasing appearance within the practical limits of terrain and area traversed is a compromise between these two extremes.
2. Vertical curvature superimposed on horizontal curvature, or vice versa, generally results in a more pleasing facility, but such combinations should be analyzed for their effect on traffic. Successive changes in profile not in combination with horizontal curvature may result in a series of humps visible to the driver for some distance which represents an undesirable condition.
3. Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition is undesirable because the driver may not perceive the horizontal change in alignment, especially at night. The disadvantages of this arrangement are avoided if the horizontal curvature leads the vertical curvature, i.e., the horizontal curve is made longer than the vertical curve. Suitable designs can also be developed by using design values well above the appropriate minimum values for the design speed.

4. Sharp horizontal curvature should not be introduced near the bottom of a steep grade approaching or near the low point of a pronounced sag vertical curve. Because the view of the road ahead is foreshortened, any horizontal curvature other than a very flat curve assumes an undesirable, distorted appearance. Further, vehicle speeds, particularly for trucks, are often high at the bottom of grades and erratic operations may result, especially at night.
5. On two-lane roads and streets, the need for passing sections at frequent intervals and including an appreciable percentage of the length of the roadway often supersedes the general guidelines for combinations of horizontal and vertical alignment. In such cases, it is appropriate to work toward long tangent sections to assure sufficient passing sight distance in design.
6. Horizontal curvature and profile should be made as flat as practical at intersections and at railroad-highway grade crossings where sight distance along both roads or streets is important and vehicles may have to slow or stop.
7. On divided highways and streets, variation in width of median and the use of independent profiles and horizontal alignments for the separate one-way roadways are desirable. Where traffic justifies provision of four lanes, a superior design without additional cost generally results from such practices.
8. In residential areas, the alignment should be designed to minimize nuisance to the neighborhood. Generally, a depressed facility makes a highway less visible and less noisy to adjacent residents. Minor horizontal adjustments can sometimes be made to increase the buffer zone between the highway and clusters of homes.
9. The alignment should be designed to enhance scenic views of the natural and manmade environment, such as rivers, rock formations, parks and outstanding structures. The highway should head into, rather than away from, those views that are outstanding; it should fall toward those features of interest at a low elevation and it should rise toward those features best seen from below or in silhouette against the sky.

Coordination of horizontal and vertical alignment should begin with preliminary design, at which time adjustments in either or both can be made jointly to obtain the desired coordination. The design criteria contained in [Chapter 1](#) and the elements of design covered in this Chapter should be kept in mind. Design speed may require adjustment during the process to conform to variations in speeds of operation due to changes in alignment characteristics needed to accommodate unusual terrain, railroad-highway grade crossings or right-of-way controls. All aspects of terrain, traffic operation and appearance should be considered and the horizontal and vertical lines should be adjusted and coordinated before the calculations and the preparation of construction plans to large scale are started.

For highways with gutters, the effects of superelevation transitions on gutter line profiles should be examined. This can be particularly significant when flat grades are involved and can result in local depressions. Slight shifts in profile in relation to horizontal curves can sometimes eliminate the problem.

For additional information on the controls and general considerations for the combination of horizontal and vertical alignment, refer to the section "Combinations of Horizontal and Vertical Alignment" in the 2004 AASHTO Green Book, Chapter 3.

2.4 SIMPLE CURVE COMPUTATIONS

The changes in direction along a highway are basically accounted for by curves consisting of portions of a circle. The simple curve computation method shall be used for all curve computations as indicated in [Figure 2.1](#).

2.5 SURVEY AND CONSTRUCTION BASELINES

1. Where spiralled curves are utilized as indicated in [Section 2.15](#), all surveying and design computations on Survey and Construction Baselines (or Centerlines) shall be achieved utilizing spiralled curves except as indicated in item 3.a below. Referencing right-of-way in spiralled areas shall be as illustrated in [Figure 2.2](#).
2. For parallel roadways with median widths of 25 m (84 ft) or less:
 - a. One Survey and Construction Centerline, located in the center of the median, shall be used for surveying of the cross sections and the design of the roadway.
 - b. One grade profile shall be required for sections of roadway that have the same grade elevations at the "Grade Points" or a constant difference between the "Grade Points" of the two pavements.
3. For transition areas with variable width medians of 25 m (84 ft) or less:
 - a. One Survey Centerline located in the center of the median shall be used for surveying the cross sections and the referencing of the right-of-way. The surveying on the survey Centerline shall be done utilizing simple curves.
 - b. Two Construction Baselines located: on the median edges of four-lane pavements, on the pavements 3.6 m (12 ft) from the median edges of the pavements of six-lane pavements, on the pavements 7.2 m (24 ft) from the median edges of the pavements of eight-lane pavements, shall be used for the design of the roadways.
 - c. Two Grade Profiles shall be developed at the same location as the Construction Baselines.
4. For roadways with medians in excess of 25 m (84 ft):
 - a. Two Survey and Construction Baselines located: on the median edges of four-lane pavements, on the pavements 3.6 m (12 ft) from the median edges of the pavements of six-lane pavements, on the pavements 7.2 m (24 ft) from the median edges of the pavements of eight-lane pavements shall be used for surveying the cross sections, for the design of the roadways and for the referencing of the right-of-way.
 - b. Two grade profiles shall be developed at the same locations as the Survey and Construction Baselines.
5. Survey Baseline shall be established and staked for all side roads. For information on the placing of Fine Grade Stakes, see Publication 122M, *Surveying and Mapping Manual*.
6. All Survey Baselines shall be monumented as stated in Publication 122M, *Surveying and Mapping Manual*, Chapter 4, Section 4.3.

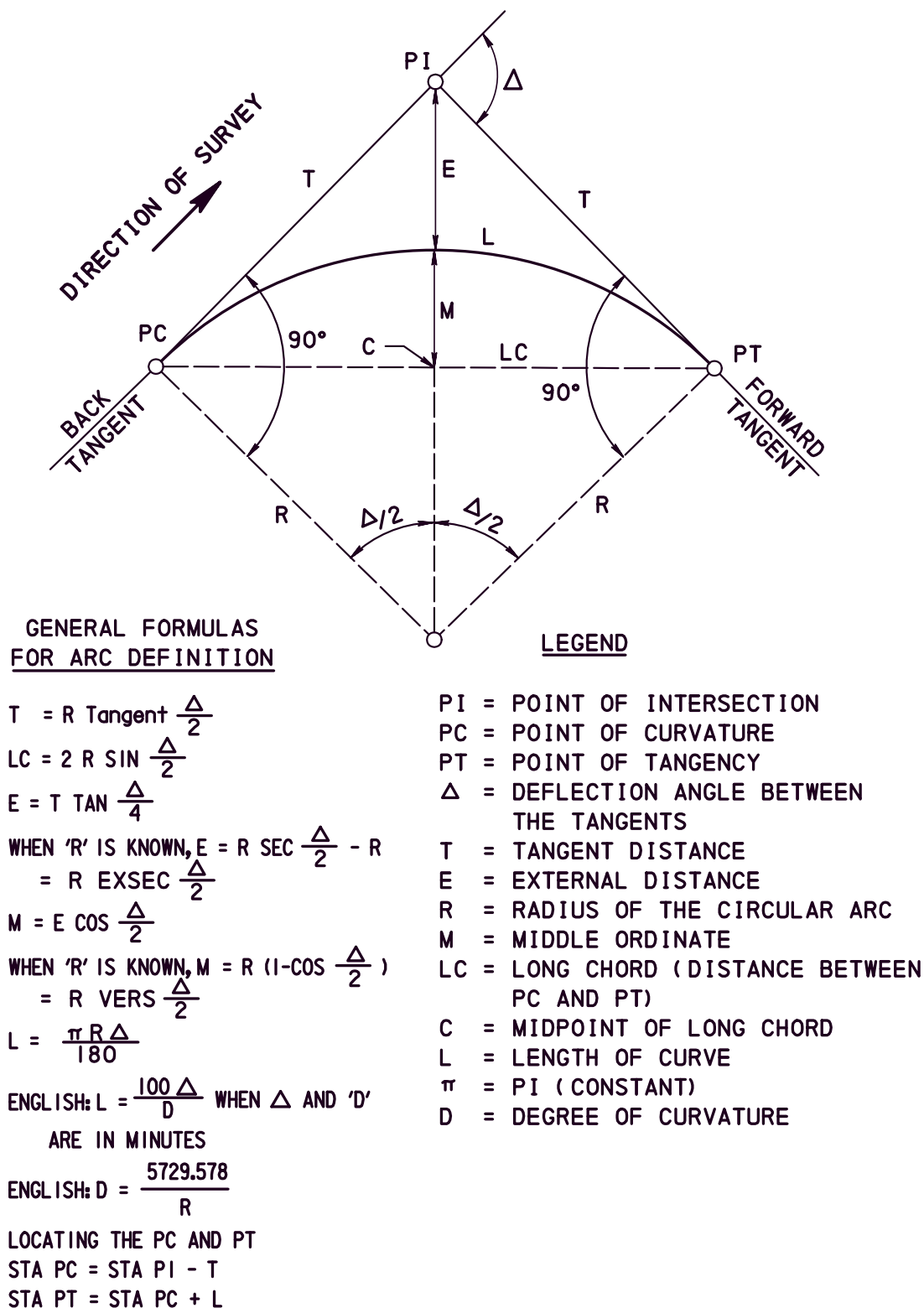
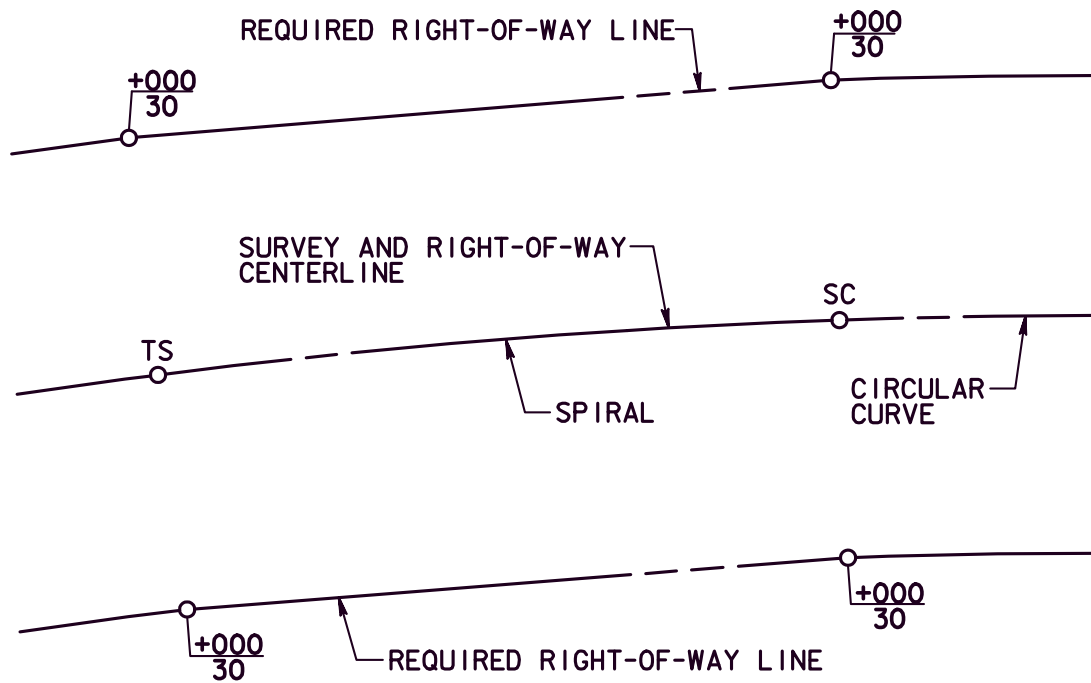
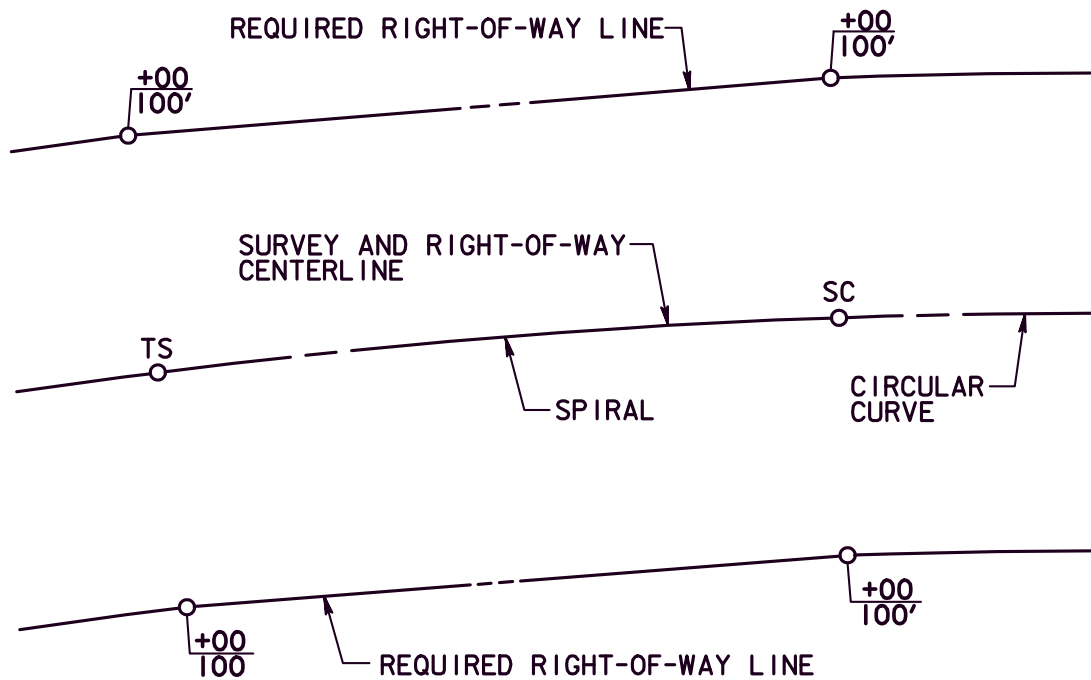


FIGURE 2.1
SIMPLE CURVE COMPUTATION METHOD



NOTE : THE RIGHT-OF-WAY LINE THROUGH THE SPIRAL AREA IS A STRAIGHT LINE. IF NECESSARY, TO CONSERVE RIGHT-OF-WAY, A SERIES OF STRAIGHT LINES MAY BE USED.

FIGURE 2.2 (METRIC)
REFERENCING RIGHT-OF-WAY IN
SPIRALLED AREAS



NOTE : THE RIGHT-OF-WAY LINE THROUGH THE SPIRAL AREA IS A STRAIGHT LINE. IF NECESSARY, TO CONSERVE RIGHT-OF-WAY, A SERIES OF STRAIGHT LINES MAY BE USED.

FIGURE 2.2 (ENGLISH)
REFERENCING RIGHT-OF-WAY IN
SPIRALLED AREAS

2.6 MINIMUM RADIUS

A. Definition. The minimum radius is a limiting value of curvature for a given design speed and is determined from the maximum rate of superelevation and the maximum allowable side friction factor. Thus, the minimum radius is a significant value in alignment design and is an important control value for the determination of superelevation rates for flatter curves.

In metric units, the minimum radius (R_{Min}) can be calculated from the following curve formula:

$$R_{Min} = \frac{V^2}{127(0.01e_{max} + f_{max})}$$

where: e_{max} = Rate of roadway superelevation (%)
 f_{max} = Side friction factor
 R_{Min} = Minimum radius (m)
 V = Design speed (km/h)

In English units, the minimum radius (R_{Min}) can be calculated from the following curve formulas:

$$R_{Min} = \frac{V^2}{15(0.01e_{max} + f_{max})}$$

where: e_{max} = Rate of roadway superelevation (%)
 f_{max} = Side friction factor
 R_{Min} = Minimum radius (ft)
 V = Design speed (mph)

B. Design for Rural Highways, Urban Freeways and High-Speed Urban Streets. The minimum radius determined for limiting values of superelevation, side friction factor and design speed is presented in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-15. The values for minimum radius shown in Exhibit 3-15 are based on maximum side friction factors recommended for rural highways (maximum superelevation rate of 8.0%) and urban highways and streets (maximum superelevation rate of 6.0%). In recognition of safety considerations, use of a maximum superelevation rate of 4.0% should be limited to urban conditions.

Utilizing less than the minimum radius results in a reduction in safety if a corresponding increase in superelevation or a reduction in design speed does not occur. When it is necessary to use less than the minimum radius, approval from the Director, Bureau of Project Delivery shall be obtained before proceeding with the design.

C. Design for Low-Speed Urban Streets. On low-speed urban streets where speed is relatively low and variable, the use of superelevation for horizontal curves can be minimized. Where side friction demand exceeds the assumed available side friction factor for the design speed, superelevation, within the range from the normal cross slope to maximum superelevation, is provided.

The 2004 AASHTO Green Book, Chapter 3, Exhibit 3-12 shows the recommended side friction factors for low-speed streets and highways as a dashed-line. These recommended side friction factors provide a reasonable margin of safety at low-speeds and lead to somewhat lower superelevation rates as compared to the high-speed friction factors. The side friction factors vary with the design speed from 0.40 at 15 km/h (0.38 at 10 mph) to 0.15 at 70 km/h (45 mph). Based on the maximum allowable side friction factors from the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-12, the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-15 gives the minimum radius for the maximum superelevation rates of 4.0%, 6.0%, and 8.0%.

For the design of horizontal curves on low-speed urban streets, drivers have developed a higher threshold of discomfort. By this design method, it is assumed that none of the lateral force is counteracted by superelevation so long as the side friction factor is less than the specified maximum for the radius of the curve and the design speed.

The equation defined in [Section 2.6.A](#) would also apply for determining the maximum comfortable speed on horizontal curves.

Although superelevation is advantageous for traffic operations, various factors often combine to make its use impractical in many low-speed urban areas. These factors include wide pavement areas, the need to meet the grade of adjacent property, surface drainage considerations, the desire to maintain low-speed operation, and frequency of cross streets, alleys, and driveways. Therefore, horizontal curves on low-speed urban streets are frequently designed without superelevation, sustaining the force solely with side friction. On these curves for traffic entering a curve to the left, the normal cross slope is an adverse or negative superelevation, but with flat curves the resultant friction needed to sustain the lateral force, even given the negative superelevation, is small.

For further guidance on design for low-speed urban streets, refer to the section "Design for Low-Speed Urban Streets" in the 2004 AASHTO Green Book, Chapter 3, and Exhibits 3-12, 3-16, and 3-17.

2.7 GRADES

Roadways should be designed to encourage uniform operation through the selection of a design speed in correlation with various geometric features of the road or street. To date, definite conclusions concerning the appropriate relationship of roadway grades to design speed have not been reached. The material presented in this section presents the vehicle-operating characteristics on grades and the control grades for design.

The effect of grades on passenger cars varies greatly due to the practices of passenger car drivers. It is generally accepted that nearly all passenger cars can readily negotiate grades as steep as four to five percent without an appreciable loss in speed below that normally maintained on level roadways. Operation on a three percent upgrade has only a slight effect on passenger car speeds compared to operations on the level. On steeper upgrades, speeds decrease progressively with increases in the grade. On downgrades, passenger car speeds are generally slightly higher than on level sections.

The effect of grades on truck speeds is more pronounced than on speeds of passenger cars. On upgrades, trucks generally decrease speed by seven percent or more that is dependent primarily on the length and steepness of the grade and the trucks weight/power ratio. On downgrades, trucks generally increase speed by up to about five percent. The effect of rate and length of grade on the speed of a typical heavy truck is illustrated in the 2004 AASHTO Green Book, Chapter 3, Exhibits 3-55 and 3-56, respectively. The data presented in these figures can serve as a valuable design guide to appraise the effect of trucks on traffic operations for a given set of profile conditions. The travel time and speed of trucks on grades is directly related to the weight/power ratio which is expressed in terms of gross weight and net power in units of kilograms/kilowatt (weight/horsepower). It has been found that trucks with weight/power ratios of about 120 kg/kW (200 lb/hp) provide acceptable operating characteristics and assures a minimum speed of about 60 km/h (35 mph) on a three percent upgrade.

Consideration of recreational vehicles (RV's) on grades is not as critical as consideration of trucks. However, on certain routes, such as designated recreational routes, where a low percentage of trucks may not warrant a truck climbing lane, sufficient recreational vehicle traffic may indicate a need for an additional lane.

The maximum and minimum grade controls for design are dependent on the topography and the functional classification of the highway and street and are presented in the Matrices of Design Values in [Chapter 1, Table 1.3 through Table 1.8](#). The maximum design grade should be used only infrequently; in most cases, grades should be less than the maximum design grade. A minimum grade of 0.5% may be used. Particular attention should be given to the design of storm water inlets and their spacing to keep the spread of water on the traveled way within tolerable limits.

For additional information on vehicle-operating characteristics on grades and control grades (maximum and minimum) for design, refer to the section "Grades" in the 2004 AASHTO Green Book, Chapter 3.

2.8 CRITICAL LENGTH OF GRADE

The term "critical length of grade" is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. If the desired freedom of operation is to be maintained on grades longer than critical, design adjustments such as a change in location to reduce grades or the addition of extra lanes should be considered.

To establish design values for critical lengths of grade where gradeability of trucks is the determining factor, the following data are required:

1. The size and power of a representative truck or truck combination for use as a design vehicle including gradeability data. A representative vehicle would be a loaded truck with a weight/power ratio of 120 kg/kW (200 lb/hp) with the gradeability data based on the 2004 AASHTO Green Book, Chapter 3, Exhibits 3-55 and 3-56.
2. Speed at entrance to critical length of grade. The average running speed, as related to design speed, can be used to approximate vehicle speed beginning an uphill climb subject to adjustments as approach conditions may determine.
3. Minimum speed on the grade below in which interference to following vehicles is considered unreasonable. Although no specific data are available for minimum tolerable speeds of trucks on upgrades, such minimum speeds should be in direct relation to the design speed. Minimum truck speeds of about 40 km/h to 60 km/h (25 mph to 40 mph) for the majority of highways (on which design speeds are about 60 km/h to 100 km/h (40 mph to 60 mph)) are not unreasonably annoying to following vehicles if the time interval during which they are unable to pass is not too long.

A common basis for determining critical length of grade is based on a reduction in speed of trucks below the average running speed of traffic. It is recommended that a 15 km/h (10 mph) reduction criterion be used as the general guide to determine critical lengths of grade. Identification of the critical length of grade for various percents of grade are discussed in the section "Critical Lengths of Grade for Design" in the 2004 AASHTO Green Book, Chapter 3 and may be determined from the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-59. Where recreational vehicles could be a control to determine critical length of grade, the control shall be determined from the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-60.

Steep downhill grades can also have a detrimental effect on the capacity and safety of facilities with high traffic volumes and numerous heavy trucks. Some downgrades are long and steep enough that some heavy vehicles travel at crawl speeds to avoid loss of control on the grade. Therefore, there are instances where consideration should be given to providing a truck lane for downhill traffic. Procedures have been developed in the *HCM* to analyze this situation.

The suggested design criterion to determine the critical length of grade is offered as a guideline. In some instances, the terrain or other physical controls may preclude shortening or flattening grades to meet these controls. Where the length of critical grade is exceeded, consideration should be given to providing an added uphill lane or climbing lane for slow-moving vehicles as presented in [Section 2.11](#).

2.9 DESIGN SPEED

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

The selected design speed should be consistent with the speeds that drivers are likely to expect on a given highway facility. Where a reason for limiting speed is obvious, drivers are more apt to accept lower speed operation than where there is no apparent reason. A highway of higher functional classification may justify a higher design speed than a lesser classified facility in similar topography, particularly where the savings in vehicle operation and other operating costs are sufficient to offset the increased costs of right-of-way and construction. A low design speed, however, should not be selected where the topography is such that drivers are likely to travel at high speeds. Drivers

do not adjust their speeds to the importance of the highway, but to their perception of the physical limitations of the highway and its traffic.

A. Projects with New or Modified Speed Posting. For projects on new location, or projects where the desired operating speed differs from the current posted speed on the roadway, the design speed should be selected with respect to the topography, anticipated operating speed, the adjacent land use, and the functional classification of the highway. The geometric features of the roadway should be designed appropriately, consistent with the established design speed, to encourage the appropriate operating speed.

Every effort should be made to use the most practical design speed to attain a desired degree of safety, mobility, and efficiency within the constraints of environmental quality, economics, aesthetics, and social or political impacts. Once the design speed is selected, all of the pertinent highway features should be related to it to obtain a balanced design.

B. Projects Maintaining Existing Speed Posting. For resurfacing, rehabilitation, and restoration (3R), and reconstruction projects, it may be appropriate to establish the design speed based on the existing posted speed limit upon analysis of safety, mobility, and efficiency. On expressways and Interstate facilities, it may be appropriate to set the design speed 10 km/h (5 mph) greater than the posted speed limit. On all other facilities, the selected design speed should equal the posted speed limit unless the aforementioned analysis of safety, mobility and efficiency warrants setting the design speed 10 km/h (5 mph) greater than the posted speed limit. The geometric features of the roadway should be designed consistent with the established design speed and the posted speed limit installed during construction should reflect the established design speed.

2.10 TERRAIN

The topography of the land traversed has an influence on the vertical and horizontal alignments of roadways and streets. To characterize variations, topography is separated into three classifications according to terrain which include: (1) level terrain, (2) rolling terrain and (3) mountainous terrain.

In level terrain, highway sight distances, as governed by both horizontal and vertical restrictions, are generally long or can be made to be so without construction difficulty or major expense.

In rolling terrain, natural slopes consistently rise above and fall below the road or street grade, and occasional steep slopes offer some restriction to normal horizontal and vertical roadway alignment.

In mountainous terrain, longitudinal and transverse changes in the elevation of the ground with respect to the road or street are abrupt, and benching and side hill excavations are frequently needed to obtain acceptable horizontal and vertical roadway alignment.

Terrain classifications pertain to the general character of a specific route corridor. Routes in valleys, passes, or mountainous areas that have all the characteristics of roads or streets traversing level or rolling terrain should be classified as level or rolling. In general, rolling terrain generates steeper grades, then level terrain, causing trucks to reduce speeds below those of passenger cars; mountainous terrain has even greater effects, causing some trucks to operate at crawl speeds.

2.11 CLIMBING LANES

On two-lane highways, a climbing lane can be used as an additional vehicle lane to accommodate slow moving vehicles and to improve operations on upgrades. A highway section with a climbing lane is not considered as a three-lane highway, but a two-lane highway with an added lane for vehicles moving slowly uphill so that other vehicles using the normal lane to the right of the centerline are not delayed.

A climbing lane is normally provided, as an added lane, for the upgrade direction of a two-lane highway where the grade, traffic volume and heavy vehicle volume combine to degrade traffic operations from those on the approach to the grade. On highways with low volumes, only an occasional vehicle is delayed and climbing lanes, although desirable, may not be justified economically even where the critical length of grade is exceeded.

The following three criteria, reflecting economic considerations, should be satisfied to justify a climbing lane:

1. Upgrade traffic flow rate in excess of 200 vehicles per hour.
2. Upgrade truck flow rate in excess of 20 vehicles per hour.
3. One of the following conditions exists:
 - a. A 15 km/h (10 mph) or greater speed reduction is expected for a typical heavy truck.
 - b. Level-of-Service E or F exists on the grade.
 - c. A reduction of two or more levels-of-service is experienced when moving from the approach segment to the grade.

The upgrade flow rate is determined by multiplying the predicted or existing design hour volume by the directional distribution factor for the upgrade direction and dividing the result by the peak hour factor. The number of upgrade trucks is obtained by multiplying the upgrade flow rate by the percentage of trucks in the upgrade direction.

On Interstate highways with ascending grades which exceed the critical design length, a climbing lane analysis should be performed and climbing lanes added where appropriate.

In addition to evaluating speed reduction, the Level-of-Service should be considered from the standpoint of highway capacity to justify the inclusion of a climbing lane. Also, safety considerations may justify the addition of a climbing lane regardless of grade or traffic volumes. For additional information on the principal determinants of need and the applicable criteria and detailed methodology for the inclusion of climbing lanes, refer to the section "Climbing Lanes" in the 2004 AASHTO Green Book, Chapter 3 and the *HCM*.

2.12 VERTICAL CURVES

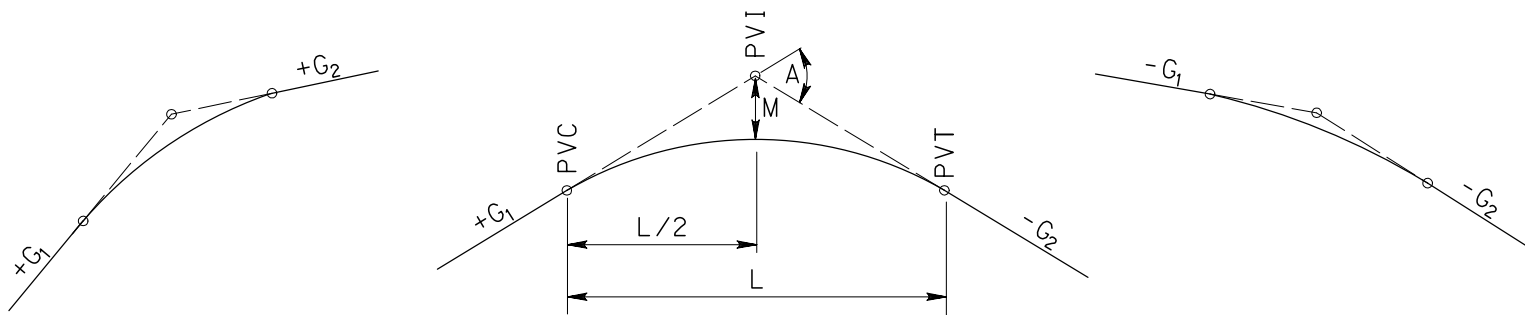
A. General Considerations. Vertical curves are used to effect gradual changes between tangent grades at their point of intersection. Vertical curves that are offset below the tangent are crest vertical curves and those offset above the tangent are sag vertical curves as shown in [Figure 2.3](#). These curves should be simple in application and should result in a design that is safe (ample sight distance), comfortable in operation (proper rate of change of grade), adequate for drainage and exhibit a pleasing appearance. The major control for safe operation on crest vertical curves is the provision of ample sight distances for the design speed. Minimum stopping sight distances should be provided in all cases. Wherever practical, more liberal stopping sight distances should be used.

For simplicity, a parabolic curve with an equivalent vertical axis centered on the point of vertical intersection (PVI) is usually used in roadway profile design. On certain occasions, because of critical clearance or other controls, the use of asymmetrical vertical curves may be appropriate. The derivation and use of the relevant equations for computing symmetrical and asymmetrical vertical curves can be found in numerous highway engineering texts.

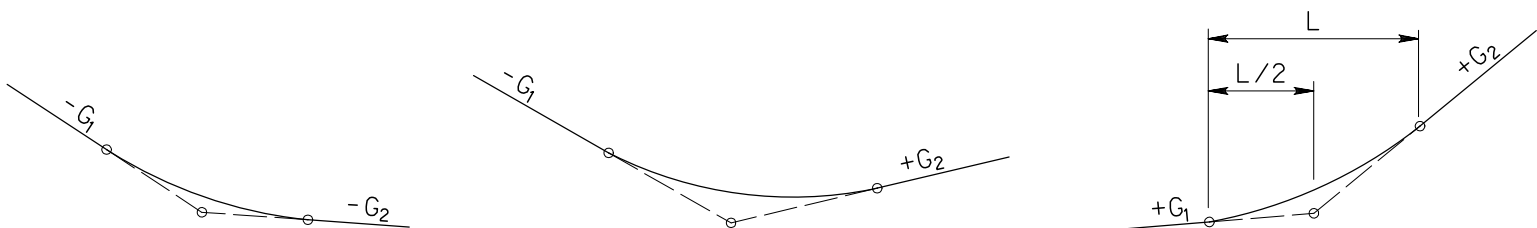
B. Crest Vertical Curves. Minimum lengths of crest vertical curves based on sight distance criteria generally are satisfactory from the standpoint of safety, comfort and appearance. An exception may be at decision areas, such as sight distance to ramp exit gores, where longer lengths are needed. For additional information concerning decision sight distance, refer to [Section 2.17](#).

The major control for safe operation on crest vertical curves is the provision of ample sight distances for the design speed. Minimum stopping sight distance should be provided in all cases as indicated in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-72. When the design speed is less than 30 km/h (20 mph), the stopping sight distances indicated in the 2004 AASHTO Green Book, Chapter 9, Exhibit 9-70 should be used. The design controls for crest vertical curves based on stopping sight and passing sight distances and the general formulas to determine minimum lengths of crest vertical curves are contained in the section "Crest Vertical Curves" in the 2004 AASHTO Green Book, Chapter 3.

G_1 AND G_2 = TANGENT GRADES (%)
 A = ALGEBRAIC DIFFERENCE IN GRADES
 L = LENGTH OF VERTICAL CURVES (m (ft))
 M = MIDDLE ORDINATE (m (ft))



CREST VERTICAL CURVES



SAG VERTICAL CURVES

FIGURE 2.3
TYPES OF VERTICAL CURVES

C. Sag Vertical Curves. At least four different criteria to establish the lengths of sag vertical curves are recognized that include: (1) headlight sight distance, (2) passenger comfort, (3) drainage control and (4) general appearance. The design controls for these curves differ from those for crest vertical curves and separate design values are required. Sag vertical curves shorter than the lengths computed may be justified for economic reasons in cases where an existing feature, such as a structure not ready for replacement, controls the vertical profile. For formulas and general design consideration for sag vertical curves refer to the section "Sag Vertical Curves" in the 2004 AASHTO Green Book, Chapter 3.

2.13 SUPERELEVATION

A. General. When a vehicle moves in a circular path, it undergoes a centripetal acceleration that acts toward the center of curvature. This acceleration is sustained by a component of the vehicle's weight related to the roadway superelevation, by the side friction developed between the vehicle's tires and the pavement surface, or by a combination of the two. The factor, known as superelevation, consists of tilting the roadway to provide safe and continuous vehicle operation. When a vehicle travels at a constant speed on a curve superelevated for that specific speed, the side friction value is zero and the centripetal acceleration is sustained by the vehicle's weight resulting in no steering effort on the part of the vehicle operator. The curves on a given facility are designed for a certain running speed and vehicles traveling at that speed should be able to negotiate the turns with ease. Vehicles, however, travel at a wide range of speeds and therefore the drivers must exert themselves to successfully negotiate these curves. They are aided by side friction on the tires.

From the above, it is evident that superelevation is predicated on design speed; therefore, the classes of highways shall be superelevated according to their speed rather than using a superelevation for a single radius for all design speeds.

B. Rates of Superelevation. The minimum and maximum cross slopes for the various functional classes of roadways are presented in the Matrices of Design Values found in [Chapter 1, Table 1.3 through Table 1.8](#). The rates of superelevation are based on specific design speeds as identified in the Matrices of Design Values. To determine the rates of superelevation for various combinations of radii and design speeds, refer to [Section 2.13.D.4](#) and to the section "Design Superelevation Tables" in the 2004 AASHTO Green Book, Chapter 3.

C. Maximum Superelevation. The maximum rates of superelevation used on highways are controlled by climate conditions, terrain conditions, type of area and frequency of very slow-moving vehicles. Consistent with current practice, the maximum rate of superelevation is 8.0%. This rate is based upon consideration of ice and snow factors and is adopted to minimize slipping across the highway by stopped vehicles or vehicles attempting to start slowly from a stopped position. A maximum rate of superelevation of 6.0% may be used in urban areas where traffic congestion or extensive marginal development acts to restrict top speeds. Where traffic congestion or extensive marginal development acts to restrict top speeds, it is common practice to utilize a low maximum rate of superelevation, usually 4.0% to 6.0%. Similarly, either a low maximum rate of superelevation or no superelevation is employed within important intersection areas or where there is a tendency to drive slowly because of turning and crossing movements, warning devices and signals. In these areas, it is difficult to warp crossing pavements for drainage without providing negative superelevation for some turning movements.

D. Superelevation Transition (T). When a motor vehicle enters or leaves a circular horizontal curve, the vehicle generally follows a suitable transition path within the limits of normal lane width. However, combinations of high speed and sharp curvature lead to longer transition paths, which can result in shifts in lateral position and encroachment on adjoining lanes. To meet the requirements of comfort and safety, incorporation of transition curves between tangents and sharp circular curves and between circular curves of substantially different radii may be appropriate in order to make it easier for a driver to keep his or her vehicle within its own lane. Superelevation transition (T) represents the progression of the roadway from a normal section to a fully superelevated section or vice versa (see [Figures 2.4, 2.5 and 2.6](#)).

The principal advantages to the application of superelevation transition (T) in horizontal alignment are as follows:

- Provides a natural, easy-to-follow path for drivers such that the lateral force increases and decreases gradually as the vehicle enters and leaves a circular curve, minimizing encroachment on adjacent lanes and promoting uniformity in speed.
- Provides a suitable location for the superelevation runoff.
- Facilitates the transition in width where the traveled way is widened on a circular curve. Superelevation transitions provide flexibility in accomplishing the widening of sharp curves.
- Enhances the appearance of the highway or street by reducing or eliminating the noticeable breaks in the alignment as perceived by drivers at the beginning and ending of circular curves.

Two terms are related to superelevation transition (T):

- Minimum Length of Tangent Runout (L_t). The minimum length of tangent runout (L_t) represents the general term denoting the length of highway section needed to accomplish the change in cross slope from a normal cross slope section to a section with the adverse cross slope removed or vice versa. The minimum length of tangent runout (L_t) is determined by the amount of adverse cross slope to be removed and the rate at which it is removed. This rate of removal should preferably be the same as the rate used to effect the minimum length of superelevation runoff (L_r).
- Minimum Length of Superelevation Runoff (L_r). The minimum length of superelevation runoff (L_r) represents the general term denoting the length of highway section needed to accomplish the change in cross slope from a section with adverse cross slope removed to a fully superelevated section or vice versa.

The specific methods of profile design for attaining the required superelevation for the various functional classification systems are diagrammatically illustrated in [Figure 2.4](#), [Figure 2.5](#) and [Figure 2.6](#).

Where curves of different radii join, the superelevation transition (T) shall be located entirely within the curve of the larger radius. The difference in the radii of the curves (R) shall be the radius used to determine the length of this transition.

For superelevation transitions between reverse curves (i.e., two closely spaced simple curves with deflections in opposite directions), a sufficient length of tangent must be provided. Along this tangent, a normal crown section does not need to be achieved; rather, the roadway may be continuously rotated in a plane about its axis. In this situation, the minimum length of tangent will be that needed to meet the superelevation requirements for the two curves.

The minimum length of spiral (L_s) indicated in [Figures 2.4](#) and [2.5](#) may be greater or less than the minimum length of superelevation runoff (L_r) depending on the formula and factors used. The minimum length of superelevation runoff (L_r) is applicable to all superelevated curves and the values for L_r may be used for the minimum lengths of spiral required.

1. Minimum Length of Superelevation Runoff. For appearance and comfort, the length of superelevation runoff should be based on a maximum acceptable difference between the longitudinal grades of the axis of rotation and the edge of traveled way. This relationship is defined as the maximum relative gradient. The axis of rotation is generally represented by the alignment centerline for undivided roadways (see [Figure 2.4](#)); however, other pavement reference lines can be used (see [Figures 2.5](#) and [2.6](#)).

The maximum relative gradient is varied with design speed to provide longer runoff lengths at higher speeds and shorter lengths at lower speeds. The 2004 AASHTO Green Book, Chapter 3, Exhibit 3-30 provides the values for the maximum relative gradients. Runoff lengths determined on this basis are directly proportional to the total superelevation, which is the product of the lane width and superelevation rate.

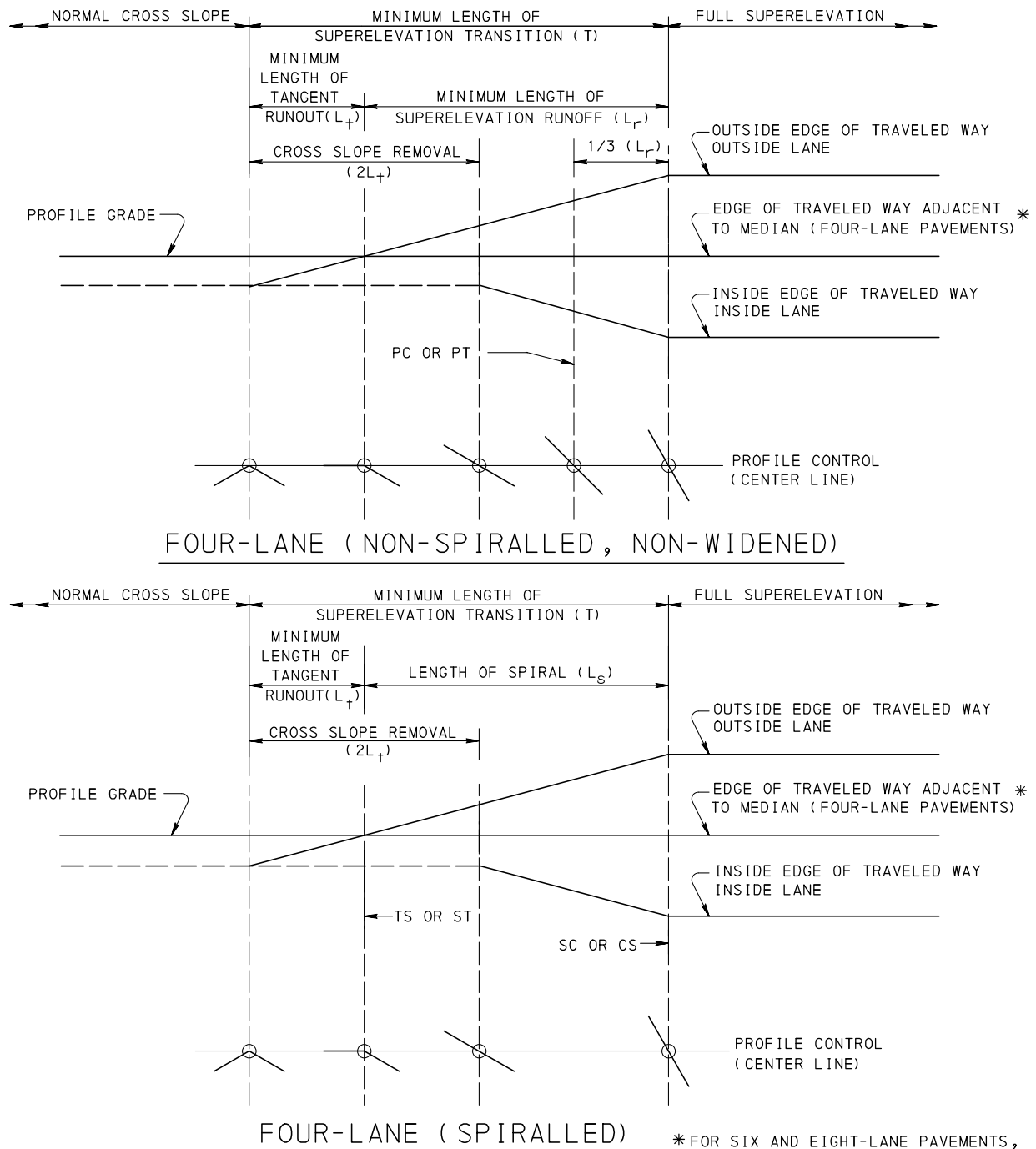


FIGURE 2.4

PROFILES SHOWING METHOD OF ATTAINING SUPERELEVATION FOR INTERSTATE AND NON-INTERSTATE LIMITED ACCESS FREEWAYS (NOTE: ALSO SEE 2004 AASHTO GREEN BOOK, EXHIBIT 3-40B FOR PROFILE CONTROL AT INSIDE EDGE OF TRAVELED WAY)

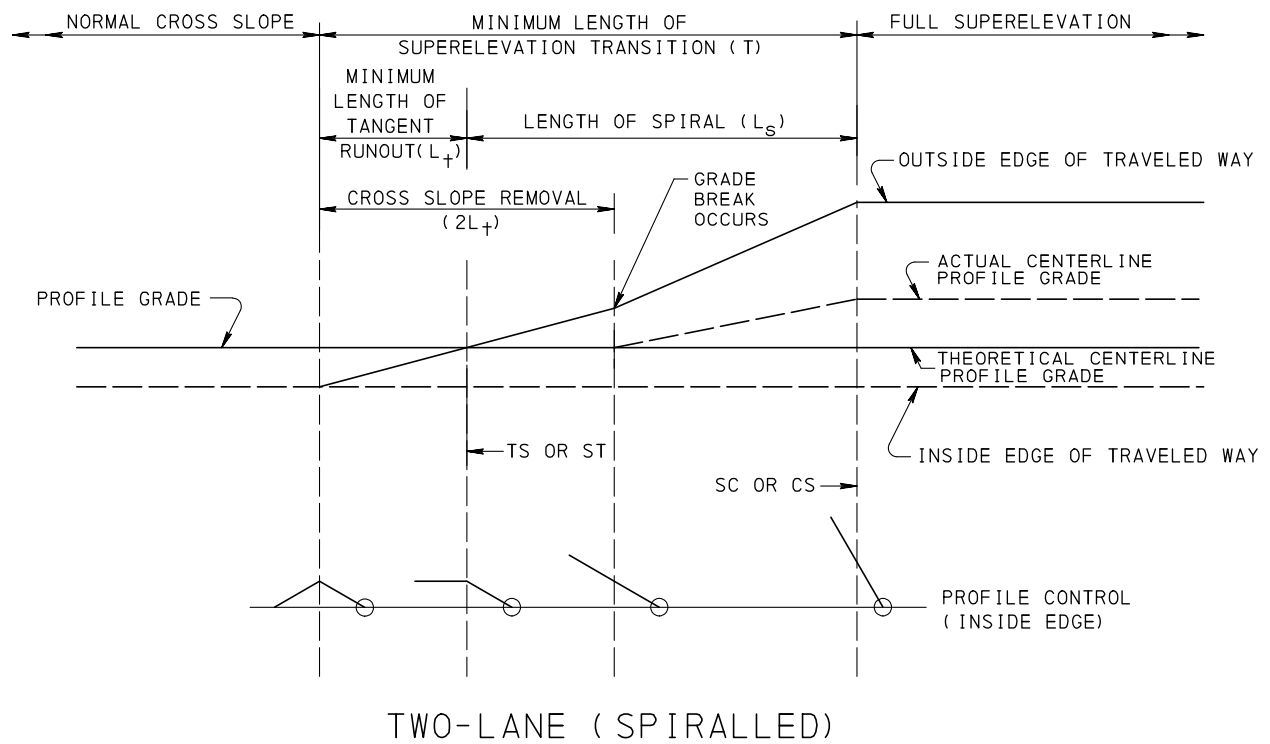
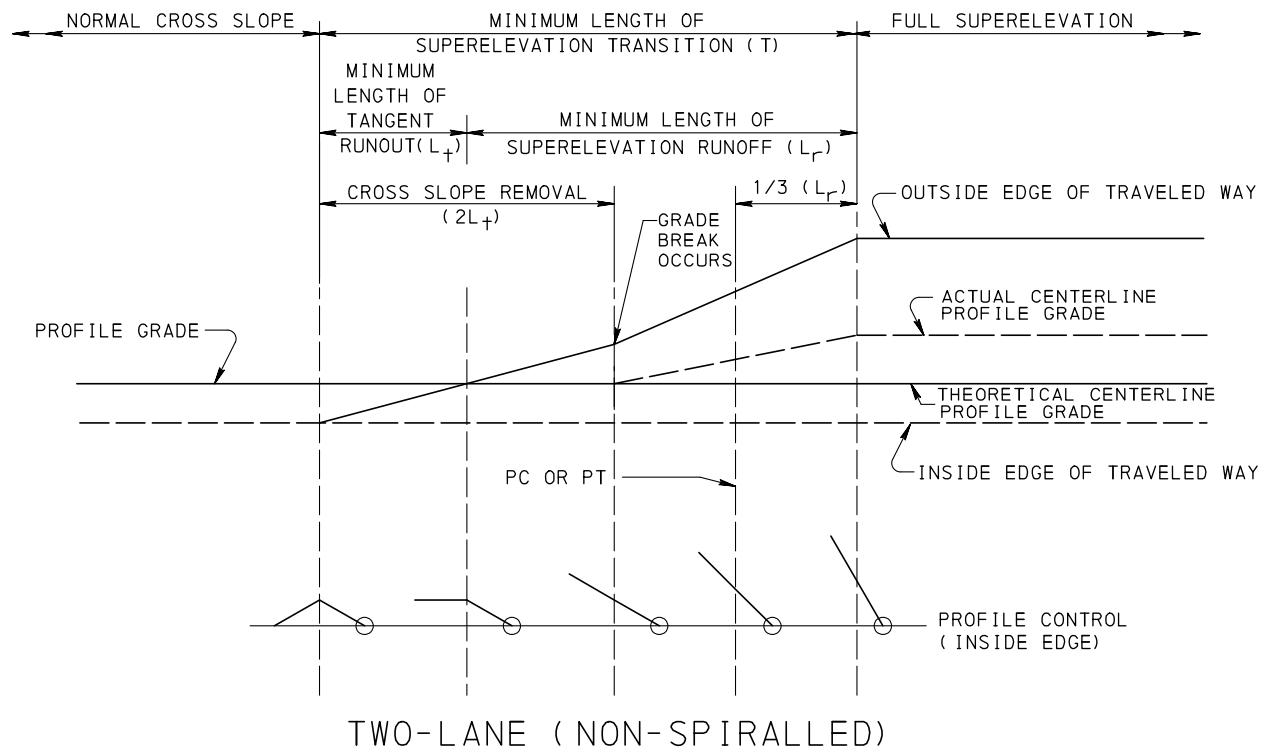
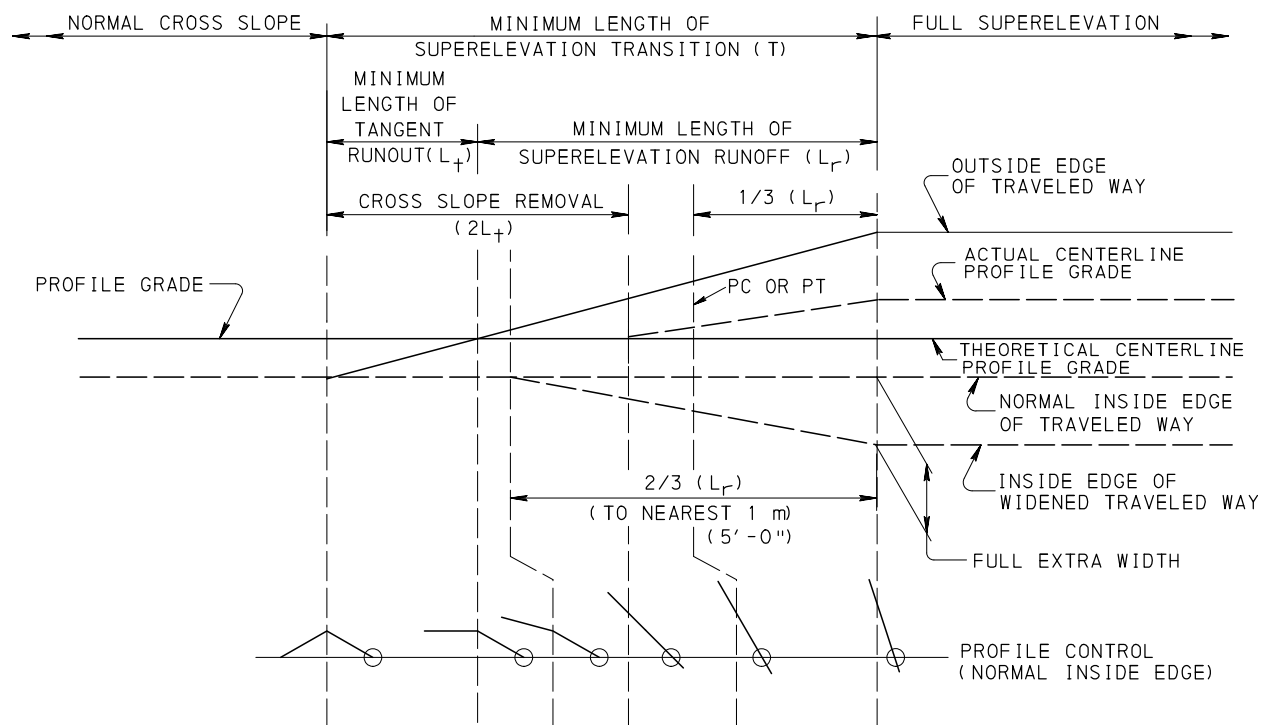


FIGURE 2.5
PROFILES SHOWING METHOD
OF ATTAINING SUPERELEVATION
FOR ARTERIALS



TWO-LANE (NON-SPIRALLED, WIDENED)

USE ARTERIAL METHOD OF ATTAINING SUPERELEVATION AS INDICATED IN FIGURE 2.5.

TWO-LANE (NON-SPIRALLED, NON-WIDENED)

FIGURE 2.6
PROFILES SHOWING METHOD
OF ATTAINING SUPERELEVATION
FOR COLLECTORS AND LOCAL ROADS

On the basis of the preceding discussion, the minimum length of superelevation runoff should be determined as:

$$L_r = \frac{(wn_1)e_d}{\Delta}(b_w)$$

where: L_r = minimum length of superelevation runoff (m (ft))
 w = width of one traffic lane (typically 3.6 m (12 ft))
 n_1 = number of lanes rotated
 e_d = design superelevation rate (percent)
 b_w = adjustment factor for number of lanes rotated
 Δ = maximum relative gradient (percent)

This equation can be used directly for undivided streets or highways where the cross section is rotated about the highway centerline and n_1 is equal to one-half the number of lanes in the cross section. More generally, this equation can be used for rotation about any pavement reference line provided that the rotated width (wn_1) has a common superelevation rate and is rotated as a plane.

A strict application of the maximum relative gradient criterion provides runoff lengths for four-lane undivided roadways that are double those for two-lane roadways; those for six-lane undivided roadways would be tripled. While lengths of this order may be considered desirable, it is often not practical to provide such lengths in design. On a purely empirical basis, the minimum superelevation runoff lengths should be adjusted downward to avoid excessive lengths for multilane roadways. The recommended adjustment factors are presented in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-31.

The adjustment factors listed in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-31 are directly applicable to undivided streets and highways. Development of runoff for divided highways is discussed in more detail in the section "Axis of Rotation with a Median" in the 2004 AASHTO Green Book, Chapter 3.

Values for the minimum length of superelevation runoff (L_r) are presented in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-32. The values for the minimum length of superelevation runoff (L_r) may be increased to provide smoother transitions. If the minimum length of superelevation runoff (L_r) is increased, a revised length of tangent runout (L_t) is required to maintain a smooth edge-of-traveled way profile.

The superelevation runoff lengths given in Exhibit 3-32 are based on 3.6 m (12 ft) lanes. For other lane widths, the appropriate runoff length should vary in proportion to the ratio of the actual lane width to 3.6 m (12 ft). Shorter lengths could be applied for designs with 3.0 m (10 ft) and 3.3 m (11 ft) lanes, but considerations of consistency and practicality suggest that the runoff lengths for 3.6 m (12 ft) lanes should be used in all cases.

2. Minimum Length of Tangent Runout. The minimum length of tangent runout is determined by the amount of adverse cross slope to be removed and the rate at which it is removed. To effect a smooth edge of traveled way profile, the rate of removal should equal the relative gradient used to define the superelevation runoff length. Based on this rationale, the following equation should be used to compute the minimum length of tangent runout:

$$L_t = \frac{e_{NC}}{e_d} L_r$$

where: L_t = minimum length of tangent runout (m (ft))
 e_{NC} = normal cross slope rate (percent)
 e_d = design superelevation rate (percent)
 L_r = minimum length of superelevation runoff (m (ft))

3. Minimum Length of Superelevation Transition. The minimum length of superelevation transition is determined by the following equation:

$$T = L_r + L_t$$

where: T = minimum length of superelevation transition (m (ft))
 L_r = minimum length of superelevation runoff (m (ft))
 L_t = minimum length of tangent runout (m (ft))

The 2004 AASHTO Green Book, Chapter 3, Exhibit 3-32 indicates the values to be applied for the minimum length of superelevation runoff (L_r). The values for the minimum length of superelevation runoff (L_r) may be increased to provide smoother transitions. If the minimum length of superelevation runoff (L_r) is increased, a revised length of tangent runout (L_t) is required to maintain a smooth edge-of-traveled way profile.

4. Design Superelevation Tables. The 2004 AASHTO Green Book, Chapter 3, Exhibits 3-25, 3-26 and 3-27 show minimum values of radius (R) for various combinations of superelevation and design speeds for each of three values of maximum superelevation rate (i.e., for a full range of common design conditions). The maximum superelevation rates are $e_{Max} = 4.0\%$ (Exhibit 3-25), $e_{Max} = 6.0\%$ (Exhibit 3-26), and $e_{Max} = 8.0\%$ (Exhibit 3-27).

Spirals are seldom used when the design superelevation rate is less than 3.0%.

When using one of the Exhibits for a given radius, interpolation is not necessary as the superelevation rate should be determined from a radius equal to, or slightly smaller than, the radius provided in the Exhibit. The result is a superelevation rate that is rounded up to the nearest 0.2%.

Found below are two examples that demonstrate how to obtain the superelevation rate for a given horizontal curve:

- a. Example 1 (Metric):** Design Speed of Horizontal Curve, $V_d = 80$ km/h
 Maximum Superelevation Rate, $e_{Max} = 8.0\%$
 Radius of Horizontal Curve, $R = 436.595$ m

Solution: From Exhibit 3-27, $e = 6.2\%$ when $R = 445$ m
 $e = 6.4\%$ when $R = 422$ m
 Determine the superelevation rate of the actual horizontal curve from the radius in Exhibit 3-27 that is equal to or slightly smaller. Since $422 \text{ m} < 436.595 \text{ m}$, specify a design superelevation rate (e_d) of 6.4%.

Example 1 (English): Design Speed of Horizontal Curve, $V_d = 50$ mph
 Maximum Superelevation Rate, $e_{Max} = 8.0\%$
 Radius of Horizontal Curve, $R = 1432.39$ ft

Solution: From Exhibit 3-27, $e = 6.2\%$ when $R = 1480$ ft
 $e = 6.4\%$ when $R = 1400$ ft
 Determine the superelevation rate of the actual horizontal curve from the radius in Exhibit 3-27 that is equal to or slightly smaller. Since $1400 \text{ ft} < 1432.39 \text{ ft}$, specify a design superelevation rate (e_d) of 6.4%.

- b. Example 2 (Metric):** Design Speed of Horizontal Curve, $V_d = 100$ km/h
 Maximum Superelevation Rate, $e_{Max} = 6.0\%$
 Radius of Horizontal Curve, $R = 1164.253$ m

Solution: From Exhibit 3-26, $e = 3.8\%$ when $R = 1170$ m
 $e = 4.0\%$ when $R = 1090$ m
 Determine the superelevation rate of the actual horizontal curve from the radius in Exhibit 3-26 that is equal to or slightly smaller. Since $1090 \text{ m} < 1164.253 \text{ m}$, specify a design superelevation rate (e_d) of 4.0%.

Example 2 (English): Design Speed of Horizontal Curve, $V_d = 60$ mph
 Maximum Superelevation Rate, $e_{Max} = 6.0\%$
 Radius of Horizontal Curve, $R = 3819.72$ ft

Solution: From Exhibit 3-26, $e = 3.6\%$ when $R = 3940$ ft
 $e = 3.8\%$ when $R = 3650$ ft
 Determine the superelevation rate of the actual horizontal curve from the radius in Exhibit 3-26 that is equal to or slightly smaller. Since $3650 \text{ ft} < 3819.72 \text{ ft}$, specify a design superelevation rate (e_d) of 3.8% .

5. Example Superelevation Problem. Found below is a superelevation problem that demonstrates how to obtain the minimum length of superelevation runoff (L_r), the minimum length of tangent runout (L_t), and the resulting length of superelevation transition (T). The data to be used in this superelevation problem comes from Example 2 in the previous subsection.

EXAMPLE SUPERELEVATION PROBLEM (METRIC)

Given: Normal Cross Slope, $e_{NC} = 2.0\%$ Find: (a) Minimum Length of Superelevation Runoff (L_r)
 $V_d = 100$ km/h (b) Minimum Length of Tangent Runout (L_t)
 $R = 1164.253$ m (c) Superelevation Transition (T)
 $e_d = 4.0\%$
 $e_{Max} = 6.0\%$
 Two 3.6 m Lanes (Non-Spiralled,
 Non-Widened)

Solution: (a) From Exhibit 3-32, $L_r = 33$ m (assuming one lane is rotated)
 (b) From equation,

$$L_t = \frac{e_{NC}}{e_d} L_r = \left(\frac{0.02}{0.04} \right) (33) = 16.5 \text{ m}$$

(c) From equation,

$$T = L_r + L_t = 33 \text{ m} + 16.5 \text{ m} = 49.5 \text{ m}$$

EXAMPLE SUPERELEVATION PROBLEM (ENGLISH)

Given: Normal Cross Slope, $e_{NC} = 2.0\%$ Find: (a) Minimum Length of Superelevation Runoff (L_r)
 $V_d = 60$ mph (b) Minimum Length of Tangent Runout (L_t)
 $R = 3819.72$ ft (c) Superelevation Transition (T)
 $e_d = 3.8\%$
 $e_{Max} = 6.0\%$
 Two 12 ft Lanes (Non-Spiralled,
 Non-Widened)

Solution: (a) From Exhibit 3-32, $L_r = 101$ ft (assuming one lane is rotated)
 (b) From equation,

$$L_t = \frac{e_{NC}}{e_d} L_r = \left(\frac{0.02}{0.038} \right) (101) = 53.16 \text{ ft}$$

(c) From equation,

$$T = L_r + L_t = 101 \text{ ft} + 53.16 \text{ ft} = 154.16 \text{ ft}$$

E. Application of Superelevation.

1. **Methods of Attaining Superelevation.** On spiral curves, the superelevation shall be applied by removing the adverse cross slope through the minimum length of tangent runout (L_t) distance to the tangent to spiral (TS). Total superelevation shall be attained along the length of spiral and held from the spiral to curve (SC) to the curve to spiral (CS).

On non-spiralled, non-widened curves, the superelevation shall be applied by using the method shown in [Figure 2.5](#). Total superelevation shall be reached at a point beyond the point of curvature (PC) at a distance equal to one-third ($1/3$) of the minimum length of superelevation runoff (L_r). The same procedure shall be followed at the point of tangency (PT).

On non-spiralled, widened curves, the superelevation shall be applied by using the method shown in [Figure 2.6](#). Curve widening shall be placed on the inside edge of curve and shall be attained in a distance equivalent to two-thirds ($2/3$) of the length of superelevation runoff (L) to the nearest 1 m (5 ft). Total superelevation and full extra width shall be reached at a point beyond the point of curvature (PC) at a distance equivalent to one-third ($1/3$) of the length of superelevation runoff (L). The same procedure shall be followed at the point of tangency (PT).

2. **Location of Profile Grade.** The profile grade line controls the roadway's vertical alignment through the horizontal curve. Although shown as a horizontal line in [Figures 2.4](#), [2.5](#) and [2.6](#), the profile grade line may correspond to a tangent, a vertical curve, or a combination of the two. In [Figure 2.4](#), the profile grade line corresponds to the centerline profile. In [Figures 2.5](#) and [2.6](#), the profile grade line is represented as a "theoretical" centerline profile as it does not coincide with the axis of rotation.

For four-lane pavement with paved or unpaved divisor areas, hold profile grade on the edge of traveled way adjacent to the divisor area, as shown in [Figure 2.4](#). See the 2004 AASHTO Green Book, Exhibit 3-40B for profile control at the inside edge of traveled way.

For six-lane pavement with paved or unpaved divisors, hold profile grade on the traveled way 3.6 m (12 ft) from the median edges of the traveled way. For eight-lane pavements with paved or unpaved divisors, hold profile grade on the traveled way 7.2 m (24 ft) from the median edges of the traveled way.

3. **Additional Design Considerations.** In the design of divided highways, the inclusion of a median in the cross section influences the superelevation transition design. This influence stems from the several possible locations for the axis of rotation. The most appropriate location for this axis depends on the width of the median and its cross section. For a discussion of common combinations of these factors and the appropriate corresponding axis location, refer to the section "Axis of Rotation with a Median" in the 2004 AASHTO Green Book, Chapter 3.

For narrow medians less than 6.0 m (20 ft) wide with concrete median barrier, special attention is needed to assure that the centerline elevations are equal elevations along curved roadway sections. Depending on the superelevation and shoulder slopes, it may be necessary to define the profile grade along the high side of the superelevation. At the same location, the low side of the superelevation would be defined as a graphic grade lower than and relative to the high side of the superelevation. For this design situation, the goal would be to install a standard concrete median barrier because it would: (1) provide the same elevations for the gutter lines on either side of the barrier; and (2) be less costly and time consuming than developing a specially-designed and constructed bifurcated concrete median barrier to accommodate differences in elevations on either side of the barrier.

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Special superelevation design shall be applied in areas involving crossover pavements to prevent flat areas and provide adequate drainage. Narrow medians may present a special problem on superelevated curves. In order to provide required shoulder slopes, it may be necessary to adjust the profile grade lines.

F. Superelevation of City Streets. Local city streets on the highway system are not generally superelevated. In built-up areas, it is desirable to discourage speed and the elimination of superelevation contributes to this objective. Also, established street grades, intersections, curbs, effect on adjacent properties and drainage conditions may inhibit the application of superelevation. There are, however, many occasions when it is desirable to provide superelevation on state highways in urban areas. For example, limited access freeways are superelevated.

G. Superelevation for Curves on Ramps. The section "General Ramp Design Considerations" in the 2004 AASHTO Green Book, Chapter 10 provides guidelines for the design of superelevation and cross-slope on ramps. Guidelines for the development of superelevation at free-flow ramp terminals are found in the 2004 AASHTO Green Book, Chapter 10, Exhibit 10-58.

2.14 TRAVELED WAY WIDENING ON HORIZONTAL CURVES

Vehicles negotiating horizontal curves may require increased traveled way width to make operating conditions on curves comparable to those on tangent sections. The reasons are twofold:

1. The design vehicle occupies a greater width because the rear wheels generally track inside front wheels (offtracking) in negotiating curves.
2. Drivers experience difficulty in steering their vehicles in the center of the lane.

Widening should transition gradually on the approaches to the curve to ensure a reasonably smooth alignment of the edge of the traveled way and to fit the paths of vehicles entering or leaving the curve. The principal points of concern in the design of curve widening which apply to both ends of highway curves are presented below:

1. On simple (unspiralled) curves, widening should be applied on the inside edge of the traveled way only. On curves designed with spirals, widening may be applied on the inside edge or divided equally on either side of the centerline. The final marked centerline, and desirably any central longitudinal joint, should be placed midway between the edges of the widened traveled way.
2. Curve widening should transition gradually over a length sufficient to make the whole of the traveled way fully usable. Preferably, widening should transition over the superelevation runoff length, but shorter lengths are sometimes used. Changes in width normally should be effected over a distance of 30 m to 60 m (100 ft to 200 ft).
3. The edge of the traveled way through the widening transition should be a smooth, graceful curve and a tangent transition edge should be avoided. The transition ends should avoid an angular break at the pavement edge.
4. On highway alignment without spirals, smooth and fitting alignment results from attaining widening with one-half to two-thirds of the transition length along the tangent and the balance along the curve. The inside edge of the traveled way may be designed as a modified spiral, with control points determined by the width/length ratio of a triangular wedge, by calculated values based on a parabolic or cubic curve, or by a larger radius (compound) curve. On highway alignment with spiral curves, the increase in width is usually distributed along the length of the spiral.

Traveled way widening on curves for the main roadway shall be undertaken in accordance with the details in [Figure 2.6](#). Widening is not required under the following conditions:

1. Traveled ways that are 7.2 m (24 ft) wide.
2. Interstate and Other Limited Access Freeways and Arterials.
3. Collectors and Local Roads when the radius is greater than 350 m (degree of curve is less than 5° 00').

Traveled way widening on ramps is discussed in [Chapter 4, Section 4.7](#). For additional information concerning traveled way widening on curves, refer to the section "Traveled Way Widening on Horizontal Curves" in the 2004 AASHTO Green Book, Chapter 3.

2.15 TRANSITION (SPIRAL) CURVES AND COMPUTATIONS

Transition spirals are curves which provide a gradual change in curvature from a straight to a circular path. Such an alignment is desirable because it permits vehicle operational comfort, gradually introduces superelevation, provides a transitional path to reduce the tendency to deviate from the traffic lane and enhances the appearance of the highway. On Interstate and Non-Interstate Limited Access Freeways, spirals are applicable to curves with radii of 1746.38 m (5729.58 ft) and less (with degree of curves of 1° and greater) and on Arterial roadways, spirals are applicable to curves with radii of 873.19 m (2864.79 ft) and less (with degree of curves of 2° and greater). Superelevation controls spiral lengths, tangent runouts and lengths of superelevation runoff for various radii and are presented in [Section 2.13](#). These are minimum values that may be exceeded.

The reference books for spiral curve computations are "Transition Curves for Highways" by Joseph Barnett and "Route Location and Design" by Thomas F. Hickerson, published by the United States Printing Office and McGraw-Hill Book Company, respectively. The minimum data shown for the spiral alignment shall be in accordance with Publication 14M, Design Manual, Part 3, *Plans Presentation*, Chapter 2.

The following example problem illustrates transition (spiral) curve computations using data from Barnett's book (Table II) and the spiral curve formulas contained in [Figure 2.7](#).

EXAMPLE PROBLEM (METRIC)

Given: PI Sta = 13 + 200.000
 Δ = 56° 00' 00" RT
 R_c = 1000.000 m
 L_s = 92.000 m

Spiral Curve Data:

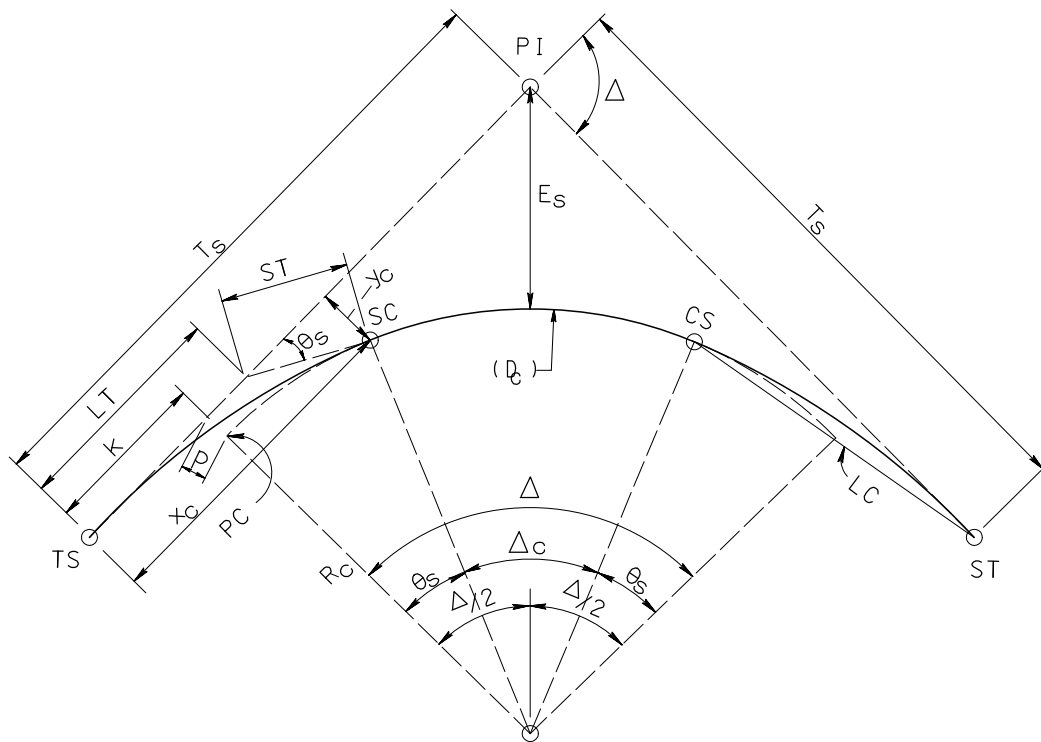
PI Sta = 13 + 200.000
 Δ = 56° 00' 00"
 Δ_c = 50° 43' 43.64"
 R_c = 1000.000 m
 L_c = 885.384 m
 θ_s = 2° 38' 08.18"
 L_s = 92.000 m
 T_s = 577.892 m
 E_s = 132.968 m
 k = 45.996 m
 p = 0.351 m
 x_c = 91.981 m
 y_c = 1.410 m
 LT = 61.340 m
 ST = 30.673 m
 LC = 91.992 m

EXAMPLE PROBLEM (ENGLISH)

Given: PI Sta = 436 + 89.20
 Δ = 56° 00' 00" RT
 D_c = 9° 00'
 R_c = 636.62'
 L_s = 300.00'

Spiral Curve Data:

PI Sta = 436 + 89.20
 Δ = 56° 00' 00"
 Δ_c = 29° 00' 00"
 D_c = 9° 00' 00"
 R_c = 636.62'
 L_c = 322.22'
 θ_s = 13° 30' 00"
 L_s = 300.00'
 T_s = 491.35'
 E_s = 91.06'
 k = 149.72'
 p = 5.88'
 x_c = 298.34'
 y_c = 23.47'
 LT = 200.58'
 ST = 100.53'
 LC = 299.26'



R_C = Radius of Circular Curve
 (D_c) = Degree of Curvature of the Circular Curve
 T_s = Tangent Distance
 Δ = Delta - External Angle
 θ_s = Spiral Angle
 Δ_c = Central Angle Between the SC and CS
 E_s = External Distance
 LC = Long Chord
 LT = Long Tangent

ST = Short Tangent
 x_C = Tangent Distance for SC
 y_C = Tangent Offset of the SC
 K = Simple Curve Co-ordinate (Abscissa)
 p = Simple Curve Co-ordinate (Ordinate)
 TS = Tangent to Spiral Point
 SC = Spiral to Curve Point
 CS = Curve to Spiral Point
 ST = Spiral to Tangent Point

FIGURE 2.7
TRANSITION SPIRAL CURVES

EXAMPLE PROBLEM (METRIC):

$$\begin{aligned}
\theta_s &= (L_s / 60.96012192) \times (1746.378852 / R_c) \\
&= (92.000 / 60.96012192) \times (1746.378852 / 1000.000) \\
\theta_s &= 2^\circ 38' 08.18'' \\
p &= ("p" \text{ constant, Table II}) \times L_s \\
&= 0.00382 \times 92.000 \\
p &= 0.351 \text{ m} \\
k &= ("k" \text{ constant, Table II}) \times L_s \\
&= 0.49996 \times 92.000 \\
k &= 45.996 \text{ m} \\
T_s &= (R_c + p) \tan \left(\frac{\Delta}{2} \right) + k \\
&= (1000.000 + 0.351) \times \tan (28^\circ 00' 00'') + 45.996 \\
T_s &= 577.892 \text{ m} \\
\Delta_c &= \Delta - 2\theta_s \\
&= 56^\circ 00' 00'' - 2 \times (2^\circ 38' 08.18'') \\
\Delta_c &= 50^\circ 43' 43.64'' \\
L_c &= (30.48006096 \times \Delta_c) / (1746.378852 / R_c) \\
&= (30.48006096 \times (50^\circ 43' 43.64'')) / (1746.378852 / 1000.000) \\
L_c &= 885.384 \text{ m} \\
\text{exsec } \frac{\Delta}{2} &= \frac{1}{\cos \frac{\Delta}{2}} - 1 \\
&= (1 / \cos (28^\circ 00' 00'')) - 1 \\
&= 0.132570 \\
E_s &= (R_c + p)(\text{exsec } \frac{\Delta}{2}) + p \\
&= (1000.000 + 0.351)(0.132570) + 0.351 \\
E_s &= 132.968 \text{ m}
\end{aligned}$$

PI Sta	13+200.000	SC Sta	12+714.108
- T _s	-577.892	+ L _c	+885.384
TS Sta	12+622.108	CS Sta	13+599.492
+ L _s	+92.000	+ L _s	+92.000
SC Sta	12+714.108	ST Sta	13+691.492

EXAMPLE PROBLEM (METRIC) (CONTINUED):

$$\begin{aligned}
 x_c &= (\text{"x" constant, Table II}) \times L_s \\
 &= 0.99979 \times 92.000 \\
 x_c &= 91.981 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 y_c &= (\text{"y" constant, Table II}) \times L_s \\
 &= 0.01533 \times 92.000 \\
 y_c &= 1.410 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 LT &= (\text{"LT" constant, Table II}) \times L_s \\
 &= 0.66674 \times 92.000 \\
 LT &= 61.340 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 ST &= (\text{"ST" constant, Table II}) \times L_s \\
 &= 0.33340 \times 92.000 \\
 ST &= 30.673 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 LC &= (\text{"LC" constant, Table II}) \times L_s \\
 &= 0.99991 \times 92.000 \\
 LC &= 91.992 \text{ m}
 \end{aligned}$$

Suppose a surveyor wanted to locate a point 50.000 m from the TS, measured along the spiral. The intersection angle (θ) between the tangent of the complete curve and the tangent at any other point on the spiral is:

$$\begin{aligned}
 \theta &= (L^2 / L_s^2) \times \theta_s \\
 &= ((50.000)^2 / (92.000)^2) \times (2^\circ 38' 08.18") \\
 \theta &= 0^\circ 46' 42.51"
 \end{aligned}$$

The values for the tangent distance (x) and tangent offset (y) are:

$$\begin{aligned}
 x &= (\text{"x" constant, Table II}) \times L \\
 &= 0.99998 \times 50.000 \\
 x &= 49.999 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 y &= (\text{"y" constant, Table II}) \times L \\
 &= 0.00453 \times 50.000 \\
 y &= 0.226 \text{ m}
 \end{aligned}$$

EXAMPLE PROBLEM (ENGLISH):

$$\begin{aligned}\theta_s &= (L_s / 200) \times D_c \\ &= (300 / 200) \times 9 \\ \theta_s &= 13^\circ 30' 00''\end{aligned}$$

$$\begin{aligned}p &= ("p" \text{ constant, Table II}) \times L_s \\ &= 0.01960 \times 300 \\ p &= 5.88'\end{aligned}$$

$$\begin{aligned}k &= ("k" \text{ constant, Table II}) \times L_s \\ &= 0.49908 \times 300 \\ k &= 149.72'\end{aligned}$$

$$\begin{aligned}T_s &= (R_c + p) \tan \left(\frac{\Delta}{2} \right) + k \\ &= (636.62 + 5.88) \times \tan (28^\circ 00' 00'') + 149.72 \\ T_s &= 491.35'\end{aligned}$$

$$\begin{aligned}\Delta_c &= \Delta - 2\theta_s \\ &= 56^\circ 00' 00'' - (2 \times (13^\circ 30' 00'')) \\ \Delta_c &= 29^\circ 00' 00''\end{aligned}$$

$$\begin{aligned}L_c &= (\Delta_c \times 100) / D_c \\ &= (29.00 \times 100) / 9 \\ L_c &= 322.22'\end{aligned}$$

$$\begin{aligned}\text{exsec} \frac{\Delta}{2} &= \frac{1}{\cos \frac{\Delta}{2}} - 1 \\ &= (1 / \cos (28^\circ 00' 00'')) - 1 \\ &= 0.132570\end{aligned}$$

$$\begin{aligned}E_s &= (R_c + p)(\text{exsec} \frac{\Delta}{2}) + p \\ &= (636.62 + 5.88)(0.132570) + 5.88 \\ E_s &= 91.06'\end{aligned}$$

PI Sta	436+89.20	SC Sta	434+97.85
- T _s	-491.35	+ L _c	+322.22
TS Sta	431+97.85	CS Sta	438+20.07
+ L _s	+300.00	+ L _s	+300.00
SC Sta	434+97.85	ST Sta	441+20.07

EXAMPLE PROBLEM (ENGLISH) (CONTINUED):

$$\begin{aligned}
 x_c &= (\text{"x" constant, Table II}) \times L_s \\
 &= 0.99446 \times 300 \\
 x_c &= 298.34'
 \end{aligned}$$

$$\begin{aligned}
 y_c &= (\text{"y" constant, Table II}) \times L_s \\
 &= 0.07823 \times 300 \\
 y_c &= 23.47'
 \end{aligned}$$

$$\begin{aligned}
 LT &= (\text{"LT" constant, Table II}) \times L_s \\
 &= 0.66862 \times 300 \\
 LT &= 200.58'
 \end{aligned}$$

$$\begin{aligned}
 ST &= (\text{"ST" constant, Table II}) \times L_s \\
 &= 0.33511 \times 300 \\
 ST &= 100.53'
 \end{aligned}$$

$$\begin{aligned}
 LC &= (\text{"LC" constant, Table II}) \times L_s \\
 &= 0.99753 \times 300 \\
 LC &= 299.26'
 \end{aligned}$$

Suppose a surveyor wanted to locate a point 152.85 ft from the TS, measured along the spiral. The intersection angle (θ) between the tangent of the complete curve and the tangent at any other point on the spiral is:

$$\begin{aligned}
 \theta &= (L^2 / L_s^2) \times \theta_s \\
 &= ((152.85)^2 / (300)^2) \times (13^\circ 30' 00") \\
 \theta &= 3^\circ 30' 16.09"
 \end{aligned}$$

The values for the tangent distance (x) and tangent offset (y) are:

$$\begin{aligned}
 x &= (\text{"x" constant, Table II}) \times L \\
 &= 0.999605 \times 152.85 \\
 x &= 152.79'
 \end{aligned}$$

$$\begin{aligned}
 y &= (\text{"y" constant, Table II}) \times L \\
 &= 0.020385 \times 152.85 \\
 y &= 3.12'
 \end{aligned}$$

INTENTIONALLY BLANK

2.16 AIRPORT - HIGHWAY CLEARANCES

The Administrator for the Federal Aviation Administration has established specific clearance requirements and criteria for highways and other structures adjacent to airports that are identified in *Part 77 of the Federal Aviation Regulations - Federal Aviation Administration*. The document is available from the Federal Aviation Administration, Washington, DC or the Superintendent of Documents, US Government Printing Office, Washington, DC and shall be used as a guide in the preparation of the design of highways adjacent or near airports to provide adequate clearance between the highways and the navigable airspace. For additional guidelines on the preparation of data, plans and other pertinent information relative to airport clearance requirements, refer to Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures*, Chapter 4, Section 4.8.F.

2.17 SIGHT DISTANCE

A. General. For safety on highways, drivers should be provided with sight distance of sufficient length to control the operation of their vehicles to avoid striking an unexpected object in the traveled way. Since the path and speed of these vehicles on highways and streets are subject to the control of drivers whose ability, training and experience are quite varied, proper sight distance shall be provided to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Sufficient sight distance should be provided on certain two-lane highways to enable drivers to occupy the opposing traffic lane for passing other vehicles without risk of a crash. Two-lane rural highways should generally provide such passing sight distance at frequent intervals and for substantial portions of their length. Sight distance represents the continuous length ahead along a roadway throughout which an object of specified height is continuously visible to the driver. In design, four sight distances are considered:

1. Passing sight distance.
2. Stopping sight distance.
3. Decision sight distance.
4. Intersection sight distance.

B. Criteria for Measuring Sight Distance. The criteria for measuring sight distances are dependent on the height of the driver's eye above the road surface, the specified object height above the road surface and the height and lateral position of sight obstructions within the driver's line of sight:

1. Height of Driver's Eye. For calculating sight distances for passenger vehicles, the height of the driver's eye above the road surface shall be considered as 1.080 m (3.5 ft). For large trucks the driver eye height shall be assumed as 2.330 m (7.6 ft) for design.
2. Height of Object. For stopping sight distance and decision sight distance calculations, the height of object shall be considered as 0.600 m (2.0 ft) above the road surface. For passing sight distance calculations, the height of object shall be considered as 1.080 m (3.5 ft) above the road surface. For intersection sight distance calculations, the height of object shall be considered as 1.080 m (3.5 ft) above the surface of the intersecting road.
3. Sight Obstructions. On a tangent roadway, the obstruction that limits the driver's sight distance is the road surface at some point on a crest vertical curve. On horizontal curves, the obstruction that limits the driver's sight distance may be the road surface at some point on a crest vertical curve, or it may be some physical feature outside of the traveled way, such as a longitudinal barrier, a bridge-approach fill slope, a tree, foliage or the backslope of a cut section. Accordingly, all highway construction plans should be checked in both the vertical and horizontal plane for sight distance obstructions.

C. Passing Sight Distance for Two-Lane Highways. Passing sight distance is the minimum sight distance that shall be available to enable the driver of one vehicle to pass another vehicle safely and comfortably, without interfering with the speed of an oncoming vehicle traveling at the design speed should it come into view after the overtaking maneuver is started. The sight distance available for passing at any place is the longest distance at which a driver whose eyes are 1.080 m (3.5 ft) above the road surface can see the top of an object 1.080 m (3.5 ft) above the road surface.

Passing sight distance for use in design should be determined on the basis of the length needed to complete normal passing maneuvers and is determined for a single vehicle passing a single vehicle. When computing minimum passing sight distance on two-lane highways, the following assumptions concerning driver behavior are made:

1. The overtaken vehicle travels at uniform speed.
2. The passing vehicle has reduced speed and trails the overtaken vehicle as it enters a passing section.
3. When the passing section is reached, the passing driver needs a short period of time to perceive the clear passing section and to react to start the maneuver.
4. Passing is accomplished under what may be termed a delayed start and a hurried return in the face of opposing traffic. The passing vehicle accelerates during the maneuver and its average speed during the occupancy of the left lane is 15 km/h (10 mph) higher than that of the overtaken vehicle.
5. When the passing vehicle returns to its lane, there is a suitable clearance length between it and an oncoming vehicle in the other lane.

The minimum passing sight distance for two-lane highways represents the sum of four elements or distances that include:

1. Distance traversed during perception and reaction time and during the initial acceleration to the point of encroachment 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).
4. Distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane (d_4 or $2/3$ of d_2 above).

These distances are diagrammatically illustrated in Exhibit 3-4 of the 2004 AASHTO Green Book, Chapter 3. For additional information concerning the components used to compute these distances, refer to the section "Passing Sight Distance for Two-Lane Highways" in the 2004 AASHTO Green Book, Chapter 3.

The values contained in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-7 represent the design values for minimum passing sight distance. These distances should be exceeded as much as practical and passing sections should be provided as often as can be done at reasonable costs to provide as many passing opportunities as practical.

Appreciable grades affect the sight distance needed for passing. The sight distances needed to permit vehicles traveling upgrade to pass safely are greater than those needed on level roads due to reduced acceleration of the passing vehicle and the likelihood that opposing traffic may speed up. Therefore, if passing maneuvers are to be performed on upgrades, passing sight distances should be greater than the derived design values in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-7. Although no specific adjustments for design are available, the desirability of exceeding the values shown should be recognized.

The frequency and length of passing sections encountered on two-lane highways depend on the topography, the design speed, the spacing of intersections and the cost. At each passing section, the length of roadway ahead, with adequate sight distance for passing equal to or greater than the passing sight distance, should be as long as practical. It is not practical to directly indicate the frequency with which passing sections should be provided on two-lane highways due to the physical and cost limitations. Where high traffic volumes are expected on a highway and a high level of service is to be maintained, frequent or nearly continuous passing sight distances should be provided.

Passing sight distance is considered only on two-lane roads. At critical locations, a stretch of four-lane construction with stopping sight distance is sometimes more economical than two lanes with passing sight distance. This is particularly practical during stage construction where two lanes of a future four-lane divided highway are being built.

The following is a summary of the design procedure to follow in providing passing sections on two-lane highways:

1. Horizontal and vertical alignment should be designed to provide as much of the highway as practical with passing sight distance (see the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-7).
2. Where the design volume approaches capacity, the effect of lack of passing opportunities in reducing the Level of Service should be recognized.
3. Where the critical length of grade exceeds the physical length of an upgrade, consideration should be given to constructing added climbing lanes. The critical lengths of grade are as shown in the 2004 AASHTO Green Book, Chapter 3, Exhibits 3-55 and 3-56.
4. Where the extent and frequency of passing opportunities made available by application of Criteria 1 and 3 are still too few, consideration should be given to the construction of passing lane sections.

D. Stopping Sight Distance. Stopping sight distance represents the length needed for a vehicle traveling at a given speed to stop before reaching an object in its path. Stopping sight distance is measured from the driver's eyes which are 1.080 m (3.5 ft) above the road surface to an object 0.600 m (2.0 ft) above the road surface. Stopping sight distance is the sum of the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop (brake reaction distance) and the distance needed to stop the vehicle from the instant brake application begins (braking distance).

The approximate braking distance of a vehicle on a level roadway may be determined from the following equation:

$$\text{METRIC: } d = 0.039 \frac{V^2}{a} \qquad \text{ENGLISH: } d = 1.075 \frac{V^2}{a}$$

where: d = braking distance (m)
 V = design speed (km/h)
 a = deceleration rate (m/s²)

where: d = Braking distance (ft)
 V = Initial speed (mph)
 a = deceleration rate (ft/s²)

Also, design speed should be used to formulate stopping distance values. The stopping sight distances presented in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-1 for the various design speeds are developed based on wet pavement conditions. Stopping sight distances exceeding those shown in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-1 should be used as the basis for design wherever practical thereby increasing the margin of safety.

The recommended stopping sight distances are based on passenger car operation. Although trucks need longer stopping distances than cars, separate stopping sight distances for both vehicles are not generally used in highway design. Because truck operators are able to see substantially farther beyond vertical sight obstructions because of the higher position of the seat in their vehicles, this factor tends to balance the additional braking lengths for trucks with those for passenger cars.

When a highway is on a grade, the equation for braking distance should be modified as follows:

$$\text{METRIC: } d = \frac{V^2}{254 \left[\left(\frac{a}{9.81} \right) \pm G \right]} \qquad \text{ENGLISH: } d = \frac{V^2}{30 \left[\left(\frac{a}{32.2} \right) \pm G \right]}$$

In this equation, G is the percent of grade divided by 100, and the other terms are as stated previously above in this section. The stopping distances needed on upgrades are shorter than on level roadways, and those available on downgrades are longer. The extent of the corrections for grade, which are based on wet pavement conditions, is indicated in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-2. On roadways traversed by traffic in both directions, the sight distance available on downgrades is larger than on upgrades, more or less automatically provides the appropriate corrections for grade.

E. Decision Sight Distance. Stopping sight distances are usually sufficient to allow reasonably competent and alert drivers to come to a hurried stop under ordinary circumstances. However, these distances are often inadequate when drivers must make complex or instantaneous decisions, when information is difficult to perceive or when unexpected or unusual maneuvers are required. Since there are many locations where it would be prudent to provide longer sight distances, decision sight distance provides the greater visibility distance that drivers need.

Decision sight distance is the distance needed for a driver to detect an unexpected or otherwise difficult-to-perceive information source or condition in a roadway environment that may be visually cluttered, recognize the condition or its threat potential, select an appropriate speed and path and initiate and complete the maneuver safely and efficiently. Decision sight distance offers drivers additional margin for error and affords them sufficient length to maneuver their vehicles at the same or reduced speed rather than to just stop.

The following are examples of critical locations where these kinds of errors are likely to occur and where it is desirable to provide decision sight distance:

1. Interchanges and intersections.
2. Locations where unusual or unexpected maneuvers are required.
3. Changes in cross section such as toll plazas and lane drops.
4. Areas of concentrated demand where there is apt to be "visual noise".
5. Railroad-highway grade crossings.

Decision sight distance criteria that are applicable to most situations have been developed from empirical data. The decision sight distances vary depending on whether the location is on a rural or urban road, and on the type of avoidance maneuver required. If it is not practical to provide these distances because of horizontal or vertical curvatures, special attention should be given to the use of suitable traffic control devices. The 2004 AASHTO Green Book, Chapter 3, Exhibit 3-3 shows decision sight distance values for various situations rounded for design.

For additional information concerning decision sight distance, refer to the section "Decision Sight Distance" in the 2004 AASHTO Green Book, Chapter 3.

F. Intersection Sight Distance. Since each intersection has the potential for several different types of vehicle conflicts, those conflicts can be greatly reduced through provisions for proper sight distances and appropriate traffic controls. The driver of a vehicle approaching an intersection should have an unobstructed view of the entire intersection, including any traffic control devices, and sufficient lengths along the intersecting highway to permit the driver to anticipate and avoid potential collisions.

For procedures to determine sight distances at intersections according to different types of traffic control, refer to the section "Intersection Sight Distance" in the 2004 AASHTO Green Book, Chapter 9. These types include:

1. Case A - Intersections with No Control.
2. Case B - Intersections with Stop Control on the Minor Road
 - a. Case B1 - Left Turn from the Minor Road
 - b. Case B2 - Right Turn from the Minor Road
 - c. Case B3 - Crossing Maneuver from the Minor Road
3. Case C - Intersections with Yield Control on the Minor Road
 - a. Case C1 - Crossing Maneuver from the Minor Road
 - b. Case C2 - Left or Right Turn from the Minor Road
4. Case D - Intersections with Traffic Signal Control
5. Case E - Intersections with All-Way Stop Control
6. Case F - Left Turns from the Major Road

Sight distance between intersecting traffic flows is not considered a requirement for intersections controlled by traffic signals, since the flows move at separate times. However, due to a variety of operational characteristics, such as violation of signal, right turn on red, malfunction of the signal etc., sight distance should be provided for signalized intersections as well. A basic requirement for all controlled intersections is that drivers must be able to see the control device soon enough to perform the action it indicates.

For additional information concerning sight distance for intersections, refer to the section "Intersection Sight Distance" in the 2004 AASHTO Green Book, Chapter 9.

G. Sight Distance for Multilane Highways. It is not necessary to consider passing sight distance on highways or streets that have two or more traffic lanes in each direction of travel. Passing maneuvers on multilane roadways are expected to occur within the limits of the traveled way for each direction of travel. Thus, passing maneuvers that involve crossing the centerline of four-lane undivided roadways or crossing the median of four-lane roadways should be prohibited. Multilane roadways should have continuously adequate stopping sight distance with greater-than-design sight distances preferred.

H. Sight Distance on Horizontal Curves. The sight distance needed across the inside of horizontal curves, where there are sight obstructions such as walls, cut slopes, buildings and possibly guide rail or median barrier, may need adjustment in the normal highway cross section or a change in the alignment if removal of the obstruction is impractical to provide adequate sight distance. Because of the many variables in alignment, in cross section and in the number, type and location of potential obstructions, specific study is usually needed for each individual curve. With sight distance for the design speed as a control, check the actual conditions on each curve and make the appropriate adjustments to provide adequate sight distance.

For general use in design of a horizontal curve, the sight line is a chord of the curve, and the stopping sight distance is measured along the centerline of the inside lane around the curve. The 2004 AASHTO Green Book, Chapter 3, Exhibit 3-53 indicates the horizontal sightline offsets for needed clear sight areas that satisfy stopping sight distance criteria presented in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-1. The horizontal sightline offset values in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-53 are derived from geometry for the several dimensions, as indicated in the diagrammatic sketch in the 2004 AASHTO Green Book, Chapter 3, Exhibit 3-54 and in the 2004 AASHTO Green Book, Chapter 3, Equation 3-38. Using the height criteria for stopping sight distance of 1.080 m (3.5 ft) for height of eye and 0.600 m (2.0 ft) for height of object, a height of 0.840 m (2.75 ft) may be used as the midpoint of the sight line where the cut slope usually obstructs sight. This assumes there is little or no vertical curvature.

The method presented in the 2004 AASHTO Green Book, Chapter 3, Exhibits 3-53 and 3-54 is only exact when both the vehicle and the sight obstruction are located within the limits of the simple horizontal curve. When either the vehicle or the sight obstruction is located beyond the limits of the simple curve, the values obtained are only approximate. The same is true if either the vehicle or the sight obstruction is located within the limits of a spiral or a compound curve. In these instances, the value obtained would result in horizontal sightline offset values slightly larger than those needed to satisfy the desired stopping sight distance.

The minimum passing sight distance for a two-lane road or street is about four times as great as the minimum stopping sight distance at the same design speed. The equation indicated in the 2004 AASHTO Green Book, Chapter 3 (Equation 3-38) is directly applicable for passing sight distance confined to tangent and very flat alignment conditions and are of limited practical value except on long curves since it is difficult to maintain passing sight distance on other than very flat curves.

2.18 OTHER ELEMENTS AFFECTING GEOMETRIC DESIGN

In addition to the design elements presented in previous Sections, there are other elements that affect or are affected by geometric design of a roadway. Each of these additional elements is addressed briefly below only to the extent necessary to indicate its relationship to geometric design.

A. Traffic Control Devices. Traffic control devices provide guidance and navigation information and also display additional information that augments some roadway or environmental feature that might otherwise be overlooked or difficult to receive. Traffic control devices include regulatory, warning, route guidance information, markings and delineation measures. Control devices are essential elements for all functional classification systems and their application shall be consistent and uniform especially at intersections where multiphase or actuated traffic signals may be needed. Geometric design should not be considered complete nor should it be implemented until it has been determined that needed traffic control devices will have the desired effect in controlling traffic.

Traffic control signals (including railroad crossing signals) represent devices that control vehicular and pedestrian traffic by assigning the right-of-way to various movements for certain pretimed or traffic-actuated intervals of time. Since they are one of the key elements in the function of many urban streets and rural intersections, the planned signal operation for each intersection of a facility should be integrated with the design so as to achieve optimum operational efficiency. The guidelines for the design and operation of traffic signals on all streets and highways shall conform to Publication 149, *Traffic Signal Design Handbook*; Publication 408, *Specifications*; Publication 111M, *Traffic Control - Pavement Marking and Signing Standards*, TC-8600 and TC-8700 Series; and Publication 148, *Traffic Standards - Signals*, TC-7800 Series. For railroad crossing signals, refer to the *MUTCD*, Part 8.

Signing and marking are directly related to the design of the highway or street and are features of traffic control and operation that should be considered in the geometric layout of such a facility. The signing and marking should be designed concurrently with the geometrics since future operational problems can be reduced significantly if both are treated as an integral part of design. The extent to which signs and markings are used depends on the traffic volume, the type of facility, and the extent of traffic control appropriate for safe and efficient operation.

Although safety and efficiency of operation depend to a considerable degree on the geometric design of the facility, the physical layout should also be supplemented by effective signing as a means of informing, warning and controlling drivers. Signing plans coordinated with horizontal and vertical alignment, sight distance obstructions, operational speeds and maneuvers and other applicable items should be worked out before completion of design.

Markings and markers, like signs, have the function of controlling traffic to encourage safe and efficient operation. Markings or markers either supplement regulatory or warning signs or serve independently to indicate certain regulations or warn of certain conditions present on the highway. The design, location and operation of all traffic signs and markings shall be governed by the regulations contained in 67 PA Code § 212.

Physical obstructions in or near the roadway should be removed in order to provide the appropriate clear zone. Where removal is impractical, such objects should be adequately marked by painting or by use of other high-visibility material. Where the object is in the direct line of traffic, the obstruction and marking thereon preferably should be illuminated at night by floodlighting; where this is not practical, the markings should be effectively reflectorized.

B. Intelligent Transportation Systems. Intelligent Transportation Systems (ITS) involve the installation and use of electronic message signs, highway advisory radios, closed circuit television (CCTV) and other electronic devices by PennDOT to provide real-time emergency or congestion information to motorists. Design guidance is found in Publication 646, *Intelligent Transportation Systems Design Guide* and Publication 647M, *Civil and Structural Standards for Intelligent Transportation Systems*. Maintenance guidance is provided in Publication 697, *Intelligent Transportation Systems Maintenance Standards*.

C. Erosion Control. Erosion prevention represents one of the major factors in design, construction and maintenance. The most direct application of erosion control occurs in drainage design and in the writing of specifications of landscaping and slope planting. Erosion and maintenance are minimized largely by: (1) the use of flat side slopes, rounded and blended with natural terrain; (2) serrated cut slopes; (3) drainage channels designed with due regard to width, depth, slopes, alignment and protective treatment; (4) inlets located and spaced with erosion control in mind; (5) prevention of erosion at culvert outlets; (6) proper facilities for groundwater

interception; (7) dikes, berms and other protective devices; (8) sedimentation devices to trap sediment at strategic locations; and (9) protective ground covers and planting. The procedures and criteria for effecting maximum erosion and sediment control shall follow the guidelines contained in [Chapter 13](#) and shall be constructed in accordance with Publication 72M, *Roadway Construction Standards* and Publication 408, *Specifications*, for all functional classification systems.

D. Landscape Development. Landscape development should be provided for aesthetic and erosion control purposes in keeping with the character of the highway and its environment. Programs include the following general areas of improvement: (1) preservation of existing vegetation; (2) transplanting of existing vegetation where practical; (3) planting of new vegetation; (4) selective clearing and thinning; and (5) regeneration of natural plant species and material.

The objectives in planting or the retention and preservation of natural growth on roadsides are closely related. In essence, they are to provide: (1) vegetation that shall be an aid to aesthetics and safety; (2) vegetation that shall aid in lowering construction and maintenance costs; and (3) vegetation that creates interest, usefulness and beauty for the pleasure and satisfaction of the traveling public.

Care should be exercised to ensure that guidelines for sight distances and clearance to obstructions are observed especially at intersections. Landscaping should also consider maintenance problems and cost, future sidewalks, utilities, additional lanes and possible bicycle facilities. All landscape development shall conform to the general principles established in [Chapter 8](#) and the additional sources of reference listed therein for all functional classification systems.

E. Railroad-Highway Grade Crossings. All railroad-highway grade crossings come under the jurisdiction of the Public Utility Commission (PUC) and cannot be constructed or altered without their prior approval. Appropriate grade crossing warning devices, as determined by the PUC, shall be installed at all railroad-highway grade crossings. Details of the available devices to be used are given in Publication 212, *Official Traffic Control Devices* and in the *MUTCD*, Part 8.

Sight distance is an important consideration at railroad-highway grade crossings. There shall be sufficient sight distance on the highway for the driver to recognize the crossing, perceive the warning device and stop if necessary.

Another important consideration is the required minimum railroad vertical and/or horizontal clearances between the track and an obstruction. Refer to Publication 371, *Grade Crossing Manual*, Appendix H for these requirements.

Additional design guidance for railroad requirements may be found in Publication 371, *Grade Crossing Manual*.

F. Drainage. Highway drainage facilities carry water across the right-of-way and remove storm water from the roadway itself. Drainage facilities include bridges, culverts, channels, curbs, gutters and various types of drains. Hydraulic design procedures, requirements for stream crossing and floodplain encroachments, hydraulic capacities and locations of the above structures should be designed to take into consideration damage to upstream property and reduce traffic interruption by flooding as is consistent with the importance of the roadway, the design traffic service requirements and available funds. The design and criteria for highway drainage facilities shall conform to the guidelines presented in [Chapter 10](#) and shall be constructed in accordance with Publication 72M, *Roadway Construction Standards*; Publication 408, *Specifications*; and other applicable Department directives. Additional design guidance may be found in Publication 584, *PennDOT Drainage Manual*.

G. Lighting. Lighting may improve the safety of a highway or street and the ease and comfort of operation thereon. Lighting of rural highways may be desirable, but the need for such fixed-source lighting is much less than on streets and highways in urban areas. Lighting of rural highways is seldom justified except in certain critical areas such as interchanges, intersections, railroad-highway grade crossings, long or narrow bridges, tunnels, sharp curves and other areas where roadside interferences are present. Since highway lighting for freeways is intimately associated with the type and location of highway signs, full effectiveness should include the joint design of these two areas. The design criteria and policies for highway lighting systems shall conform to the procedures presented in [Chapter 5](#) and the additional sources of reference listed therein and shall be constructed in accordance with Publication 72M, *Roadway Construction Standards* and Publication 408, *Specifications*.

H. Safety Rest Areas, Welcome Centers and Scenic Overlooks. These areas represent functional and desirable elements of the complete highway facility and are provided for the safety and convenience of highway users. Site selection for safety rest areas, welcome centers and scenic overlooks should consider the scenic quality of the area, accessibility and adaptability to development that includes facilities designed to accommodate the needs of older persons and persons with disabilities. The design procedures associated with these facilities shall be in accordance with the criteria presented in [Chapter 9](#) and the additional sources of reference listed therein.

I. Utilities. Highway and street improvements, whether upgraded within the existing right-of-way or entirely on new right-of-way, must be designed to avoid or minimize impacts to utility facilities. This is in accordance with State and Federal regulations (PA One Call, 23 CFR and the Federal Program Guide on Utility Relocation and Accommodation on Federal-Aid Highway Projects) and must be done.

The existing utilities should be placed on the plans early in the development of a project to identify conflicts when designing the new or upgraded highway and street improvements.

The use of Subsurface Utility Engineering (SUE) may be required to determine the exact horizontal and vertical location of all underground utilities (refer to Publication 16M, Design Manual, Part 5, *Utility Relocation*, Chapter 6, Subsurface Utility Engineering). The SUE Utility Impact Form (see Publication 16M, Design Manual, Part 5, *Utility Relocation*, Appendix A-501) is a tool that was developed to address the legal requirements and to comply with the State and Federal laws. The form provides an analysis based on project criteria to determine if SUE use is "practicable," when SUE should be considered on a project, and what SUE quality levels should be utilized. SUE should be considered for all projects regardless of a project's estimated cost. The SUE Impact form should be completed by the Project Manager in coordination with the District Utility Relocation Unit.

Special construction items should be shown on the utility submissions (see Publication 16M, Design Manual, Part 5, *Utility Relocation*, Appendix A-502).

It is important that the Project Manager coordinate plan revisions with the District Utility Administrator to determine the effect they may have on the relocation of utilities. In many cases minor revisions to the highway plan can have major impacts to the relocation of utilities.

Although utilities generally have little effect on the geometric design of the highway or street, full consideration should be given to measures, reflecting sound engineering principles and economic factors, needed to preserve and protect the integrity and visual quality of the highway or street, its maintenance efficiency and the safety of traffic. The various policies and procedures to accomplish utility adjustments made necessary by highway construction projects are contained in Publication 16M, Design Manual, Part 5, *Utility Relocation*.

J. Bicycle Facilities. The bicycle has become an element for consideration in the highway design process. Most of the distance required for bicycle travel is comprised of the current street and highway system. However, at certain locations or in certain corridors, a designated bikeway (for either exclusive or nonexclusive bicycle use) may be provided to supplement the existing street or highway system. The design of bikeway facilities shall adhere to the guidelines presented in [Chapter 16](#) and [Chapter 19](#).

K. Noise Control and Noise Barriers. Since motor vehicles generate traffic noise, the design of highways may require the establishment of measures to minimize the radiation of noise into noise-sensitive areas by evaluating existing or potential noise levels and estimate the effectiveness of reducing highway traffic noise through location and design considerations. The actual noise level is not, in itself, a good predictor of annoyance since human reactions to noise are usually less if the noise source is hidden from view. The type of development in a particular area can affect the annoyance level since high traffic noise levels are usually more tolerable in industrial than in residential areas. Other factors that influence human reactions to noise are pitch and intermittency because the higher the pitch or the more pronounced the intermittency of the noise, the greater the degree of annoyance.

To combat the adverse effect noise can have on people living on, working on or otherwise using land adjacent to highways, noise barriers may be constructed on both new and existing highways. Careful consideration shall be exercised to ensure the location and construction of noise barriers shall not compromise the safety of the highway by providing proper horizontal clearances and adequate sight distances particularly where the location of the noise barrier is along the inside of a curve. An effective method of reducing traffic noise from adjacent areas is to design

the highway so that some form of solid material such as earth or concrete blocks the line of sight between the noise source and the receptors. Buffer plantings such as shrubs, trees or ground covers offer some noise reduction while exceptionally wide and dense plantings may result in substantial reductions in noise levels. In terms of noise considerations, a depressed highway section is the most desirable noise reduction design. For additional information and general considerations for noise barriers including design procedures, noise reduction designs and the assessment of noise impacts on highway projects, refer to Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures*, Chapter 4, Section 4.9.G.12 and the section "Noise Control" in the 2004 AASHTO Green Book, Chapter 4.

L. Structures. The established standard policies, procedures and specifications for the design and construction of highway structures shall be governed by Publication 15M, Design Manual, Part 4, *Structures* and the additional sources of reference listed therein.

M. Pedestrian Facilities. Since pedestrians represent a part of every roadway environment, attention should be paid to their presence in rural as well as in urban areas. The urban pedestrian, being more prevalent, influences roadway design features more often than the rural pedestrian does. Pedestrian facilities may include sidewalks, crosswalks, traffic control features and curb ramps for persons with disabilities. When designing urban highways with substantial pedestrian-vehicle conflicts, the following are some measures that could be considered to help reduce these conflicts and may increase the efficient operation of the roadway: (1) eliminate left and/or right turns; (2) prohibit free-flow right-turn movements; (3) prohibit right turn on red; (4) convert from two-way to one-way street operation; (5) provide separate signal phases for pedestrians; (6) eliminate selected crosswalks; and (7) provide for pedestrian grade separations. Pedestrian accommodation and pedestrian facilities shall follow the design criteria and guidelines presented in [Chapter 6](#). For additional information concerning general considerations, physical characteristics of pedestrians and characteristics of pedestrians with disabilities, refer to the section "The Pedestrian" in the 2004 AASHTO Green Book, Chapter 2 and [Chapter 19](#).

N. Highway Capacity Analysis. The term "capacity" is used to express the maximum number of vehicles that have a reasonable expectation of passing over a given section of a lane or a roadway during a given time period under prevailing roadway and traffic conditions. The principles and major factors concerning highway design capacity analysis are presented in the *HCM*.

O. Mass Transit Facilities. Wherever there is a demand for highways to serve automobile traffic, there is likewise a demand for public transportation. The requirements for public transit and their compatibility with other highway traffic shall be considered in the development and design of highways to insure the forms of interference between the two are minimized through careful planning, design and traffic control measures. Mass transit facilities may include bus stops and bus turnouts, park and ride facilities, rail transit and high-occupancy vehicle (HOV) facilities. For additional information concerning the location and design of these mass transit facilities, refer to the sections "Bus Turnouts", "Public Transit Facilities", and "Accommodation of Transit and High-Occupancy Vehicle Facilities" in the 2004 AASHTO Green Book, Chapters 4, 7, and 8, respectively and [Chapter 19](#).

P. Special Purpose Roads. For the purpose of design, highways are classified by function with specific design criteria given for each functional classification (see [Chapter 1, Section 1.2](#)). However, certain roads do not fit into any of the current functional classifications due to their purpose and are referred to as special purpose roads. These roads include: (1) recreational roads; (2) resource recovery roads; and (3) local service roads and, because of their uniqueness, separate design criteria are provided as presented in the section "Special-Purpose Roads" in the 2004 AASHTO Green Book, Chapter 5. Recreation roads, as the name implies, serve recreation sites and areas through the use of primary access roads, circulation roads and area roads. Resource recovery roads include mining and logging roads that are primarily composed of large, slow-moving, heavily loaded vehicles. Local service roads represent roads serving isolated areas that have little or no potential for further development with traffic that is very low and generally consists of drivers who are familiar with the road.

Although not classified as special purpose roads, frontage roads, cul-de-sacs, turnarounds and alleys are presented herein since their functions apply to special areas of accessibility. Frontage roads serve numerous functions depending on the type of roadway they serve and the character of the surrounding area. Frontage roads can be used on all types of highways to: (1) control access to an arterial; (2) function as a street facility serving adjoining properties; (3) maintain circulation of traffic on each side of an arterial; (4) segregate local traffic from higher-speed, through traffic; and (5) intercept driveways of residences and commercial establishments along the highway. For information on data and features of frontage roads, refer to the section "Frontage Roads" in the 2004 AASHTO

Green Book, Chapter 4. Cul-de-sacs and turnarounds are normally employed for use on a local street system that is open at one end. A special turning area at the closed end is used to enable passenger vehicles and local delivery trucks to U-turn or at least turn around by backing once. Alleys are also associated with local street systems. They provide access to the side or rear individual land parcels with connections to streets or to other alleys. The geometric features and design guidelines for cul-de-sacs, turnarounds and alleys are presented in the sections "Cul-de-Sacs and Turnarounds" and "Alleys" in the 2004 AASHTO Green Book, Chapter 5.

Q. Traffic Barriers. Traffic barriers are used to prevent vehicles that leave the traveled way from hitting an object that has greater crash severity potential than the barrier itself. Because barriers are a source of crash potential themselves, their use should be carefully considered. The criteria and guidelines for the design, placement and installation of longitudinal barrier systems (roadside barriers and median barriers) and impact attenuators are presented in [Chapter 12](#).

R. Curbs and Driveways. The type and location of curbs affects driver behavior and, in turn, the safety and utility of a highway. Curbs serve any or all of the following purposes: (1) drainage control; (2) roadway edge delineation; (3) right-of-way reduction; (4) aesthetics; (5) delineation of pedestrian walkways; (6) reduction of maintenance operations; and (7) assistance in orderly roadside development. Curb configurations include both vertical and sloping curves. The design and construction of curbs shall be in accordance with Publication 72M, *Roadway Construction Standards*; Publication 408, *Specifications*; and [Chapters 6, 7, and 9](#). For further information refer to the sections "Curbs" and "Driveways" in the 2004 AASHTO Green Book, Chapter 4. Vertical curbs should not be used along freeways or other high-speed roadways because an out-of-control vehicle may overturn or become airborne as a result of an impact with such a curb. Since curbs are not adequate to prevent a vehicle from leaving the roadway, a suitable traffic barrier should be provided where redirection of vehicles is needed. Sloping curbs can be used at median edges to outline channelizing islands in intersection areas or at the outer edge of the shoulder. Sloping curbs are designed so vehicles can cross them readily when the need arises.

S. Maintenance of Traffic Through Construction Areas. Maintaining a safe flow of traffic during construction shall be carefully planned in the development of construction plans, and the designs for traffic control shall minimize the effect on traffic operations by minimizing the frequency or duration of interference with normal traffic flow. The development of traffic control plans is an essential part of the overall project design and depends on the nature and scope of the improvement, volumes of traffic, highway or street pattern and capacities of available highways or streets. A well-thought-out and carefully developed traffic control plan through a construction work zone can contribute significantly to the safe and efficient flow of traffic as well as the safety of the construction forces.

The goal of any traffic control plan should be to safely route traffic at a controlled speed through or around construction areas with geometrics and traffic control devices as nearly comparable to those utilized for normal operating situations as practical while providing room for construction operations. The maintenance of traffic through construction areas shall adhere to the guidelines presented in Publication 212, *Official Traffic Control Devices* and Publication 213, *Temporary Traffic Control Guidelines*.

T. Outer Separations and Border Areas. The area between the traveled way of a roadway for through traffic and a frontage road (see Section O above) or a street for local traffic is referred to as the outer separation. These function as buffers for noise abatement in sensitive areas and provide space for shoulders, sideslopes, drainage, access-control fencing and possibly retaining walls and ramps in urban areas. The outer separation should be as wide as economically possible so local traffic will have less influence on through traffic and should lend itself to landscape treatment that can enhance the appearance of both the highway and adjoining property. Where ramp connections are provided between the through roadway and the frontage road, the outer separation should be wider than normal with the needed width dependent on the design requirements of the ramp termini. Where two-way frontage roads are provided, desirably the outer separation should be sufficiently wide to minimize the effects of the approaching traffic particularly the nuisance of headlight glare at night.

The cross section and treatment of an outer separation depends largely upon its width and the type of arterial and frontage road. Preferably, the strip should drain away from the through roadway either to a curb and gutter at the frontage road or to a swale within the strip. Typical cross sections for outer separations are presented in the 2004 AASHTO Green Book, Chapter 4, Exhibit 4-13.

Where there are no frontage roads or local streets functioning as frontage roads, the area between the traveled way of the main lanes and the right-of-way line is referred to as the border area. The border area between the roadway and the right-of-way line should be wide enough to serve several purposes including provision of a buffer space between pedestrians and vehicular traffic, sidewalk space, snow storage, an area for placement of underground and above ground utilities such as traffic signals, parking meters and fire hydrants and an area for maintainable aesthetic features such as grass or other landscaping features. The border width should be 2.4 m (8 ft) wide and preferably 3.6 m (12 ft) wide or more. Every effort should be made to provide wide borders not only to serve functional needs but also as a matter of aesthetics, safety and reducing the nuisance of traffic to adjacent development.

U. Median Crossovers. Consideration shall be given to providing openings in medians on Interstate and other Limited Access Freeways for use by emergency and other authorized vehicles. Median crossovers are also intended to be used where operation of snow and ice removal equipment to clear interchange ramps would be expedited. The need for such openings shall be determined by the District Executive and their use shall conform to the guidelines presented below. Median barrier and end treatments, if required at these locations, shall be constructed as indicated on the Standard Drawings. Crossovers should never be provided unless justified. Median crossovers constructed on Interstate Highways shall be approved by the Federal Highway Administration (FHWA). A submission shall be made to the Central Office, Bureau of Project Delivery, Highway Delivery Division, Highway Design and Technology Section for this purpose.

Median crossovers can be used on Interstate and Non-Interstate Limited Access Freeways when the median width is greater than 10 m (33 ft). If crossovers are required in a median where the width is not greater than 10 m (33 ft), coordination with the Central Office, Bureau of Project Delivery, Highway Delivery Division, Highway Design and Technology Section is required. Median crossovers can be located as follows:

1. Where the distance between the ends of speed-change tapers for adjacent interchanges is less than 6 km (4 mi), one median crossover may be provided. This crossover should be constructed at a suitable location midway between the interchanges, but not closer than 450 m (1,500 ft) from the end of any speed-change taper or structure. Crossovers should be located only where above-minimum stopping sight distance is provided and preferably should not be located on superelevated curves.
2. Where the distance between the ends of speed-change tapers for adjacent interchanges is greater than 6 km (4 mi), two or more median crossovers may be provided. These crossovers shall be provided at no less than 5 km (3 mi) intervals, and shall not be constructed closer than 450 m (1,500 ft) from the end of any speed-change taper or structure.
3. One set of dual crossovers may be located at or near a State or County line if the proximity of the nearest interchange or median crossover is greater than 1.6 km (1 mi). The intent of the dual crossovers is to allow for the safe operation of winter maintenance activities, eliminating the need for winter maintenance vehicles to back up to a crossover after plowing or spreading materials beyond the crossover. If an acceptable location for dual crossovers cannot be established at the State or County line, the pair may be shifted to the nearest acceptable location that has adequate sight distance. If dual crossovers are required, coordination with the Central Office, Bureau of Project Delivery, Highway Delivery Division, Highway Design and Technology Section is required.

All median crossovers shall conform to the typical detail shown in [Figure 2.8](#). They shall be constructed with a paved surface, paved shoulders, and deceleration lanes as shown. A typical detail for dual crossovers at or near a State or County line is shown in [Figure 2.9](#).

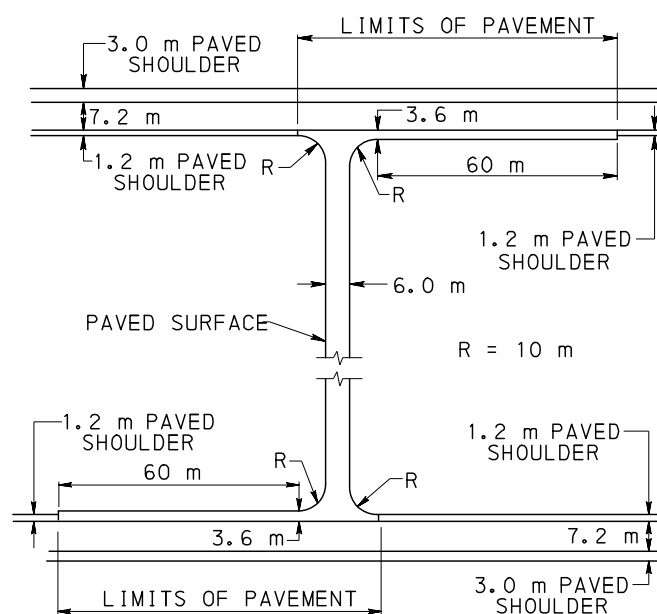
The location of median crossovers should be coordinated with proposed or existing median drainage systems to eliminate exposed pipe end sections that could present an obstacle to errant vehicles. Where cross pipes are deemed necessary, careful thought should be given to providing a safe, hydraulically-efficient drainage system that uses proper drainage appurtenances to eliminate undesirable conditions. The kind, size, and location of the drainage system required is dependent on actual field conditions.

All exposed culvert end sections shall be designed to ensure they can be safely negotiated by errant vehicles. The ends of the pipe should be sloped to match the side slopes of the crossover. All pipe openings greater than 450 mm (18 in) in diameter should be designed to provide safe traversability by using a grate, longitudinal or transverse bars

or a combination thereof. Embankment slopes in the crossover area should be 1V:6H minimum, longitudinally and 1V:12H minimum, transversely (1V:20H desirable), with respect to the crossover pavement.

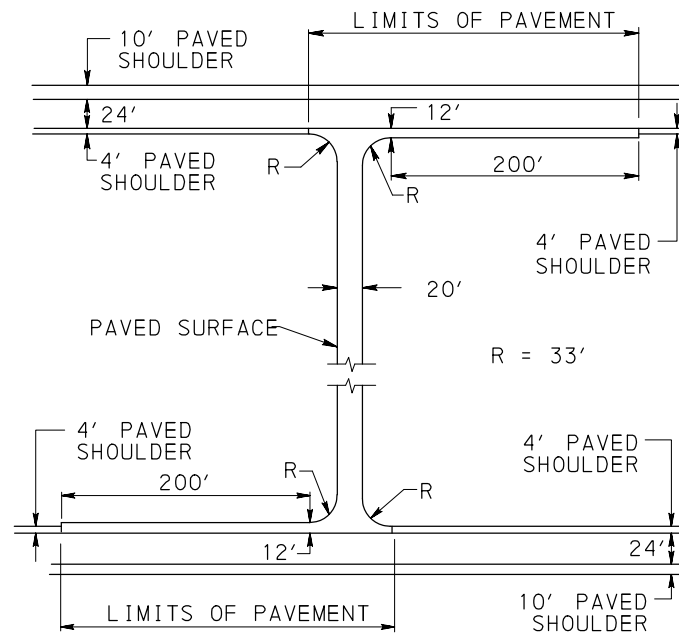
In order to limit usage to emergency and other authorized vehicles, appropriate signing and delineation shall be used (see Traffic Standard TC-8604).

When eliminating existing crossovers, proper coordination should be conducted with local emergency management officials to ensure their operations are not significantly impacted.



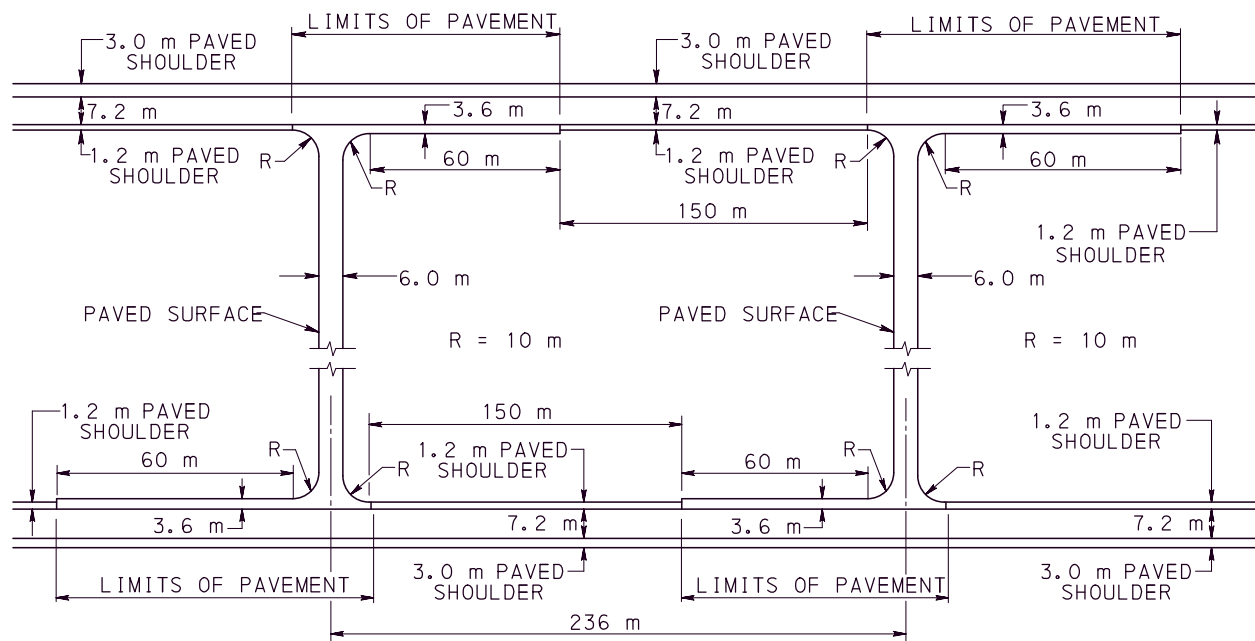
1. TRANSVERSE SLOPES SHOULD BE A MINIMUM 1V:12H WITH RESPECT TO THE CROSSOVER PAVEMENT, FLAT AND FREE OF ROADSIDE OBSTRUCTIONS.
2. LONGITUDINAL SLOPES SHOULD BE A MINIMUM 1V:6H WITH RESPECT TO THE CROSSOVER PAVEMENT, UNLESS PROTECTED BY A GUIDE RAIL.
3. PAVED SURFACE SHALL CONSIST OF: 40 mm HMA WEARING COURSE, 100 mm HMA BASE COURSE, 150 mm SUBBASE OR 40 mm HMA WEARING COURSE, 150 mm AGG-BIT, 150 mm SUBBASE.
4. WHEN CONCRETE SHOULDERS ARE PROVIDED ON THE MAINLINE, THE 1.2 m CONCRETE SHOULDER SHALL BE CONTINUED THROUGH THE DECELERATION LANE AND CROSSOVER AREA. THE ADDITIONAL 2.4 m WIDENING FOR THE DECELERATION LANE AND THE CROSSOVER PAVEMENT SHALL BE A PAVED SURFACE CONSISTING OF: 40 mm HMA WEARING COURSE, 100 mm HMA BASE COURSE, 150 mm SUBBASE OR 40 mm HMA WEARING COURSE, 150 mm AGG-BIT, 150 mm SUBBASE.

FIGURE 2.8 (METRIC)
TYPICAL PERMANENT
MEDIAN CROSSOVER



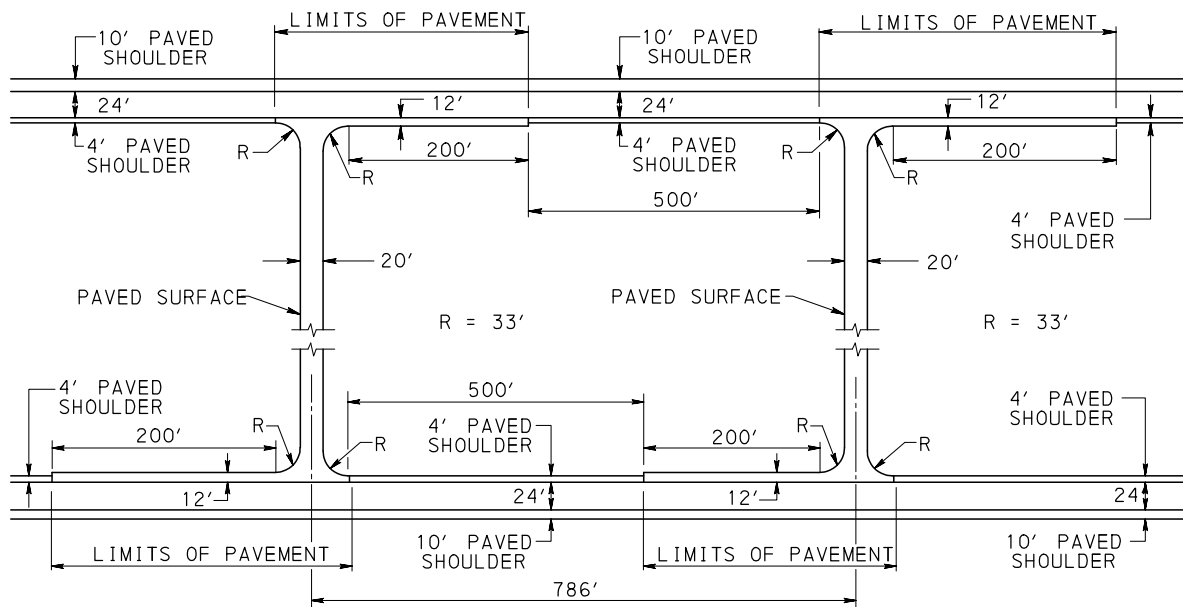
1. TRANSVERSE SLOPES SHOULD BE A MINIMUM 1V:12H WITH RESPECT TO THE CROSSOVER PAVEMENT, FLAT AND FREE OF ROADSIDE OBSTRUCTIONS.
2. LONGITUDINAL SLOPES SHOULD BE A MINIMUM 1V:6H WITH RESPECT TO THE CROSSOVER PAVEMENT, UNLESS PROTECTED BY A GUIDE RAIL.
3. PAVED SURFACE SHALL CONSIST OF: 1½" HMA WEARING COURSE, 4" HMA BASE COURSE, 6" SUBBASE OR 1½" HMA WEARING COURSE, 6" AGG-BIT, 6" SUBBASE.
4. WHEN CONCRETE SHOULDERS ARE PROVIDED ON THE MAINLINE, THE 4' CONCRETE SHOULDER SHALL BE CONTINUED THROUGH THE DECELERATION LANE AND CROSSOVER AREA. THE ADDITIONAL 8' WIDENING FOR THE DECELERATION LANE AND THE CROSSOVER PAVEMENT SHALL BE A PAVED SURFACE CONSISTING OF: 1½" HMA WEARING COURSE, 4" HMA BASE COURSE, 6" SUBBASE OR 1½" HMA WEARING COURSE, 6" AGG-BIT, 6" SUBBASE.

FIGURE 2.8 (ENGLISH)
TYPICAL PERMANENT
MEDIAN CROSSOVER



1. TRANSVERSE SLOPES SHOULD BE A MINIMUM 1V:12H WITH RESPECT TO THE CROSSOVER PAVEMENT, FLAT AND FREE OF ROADSIDE OBSTRUCTIONS.
2. LONGITUDINAL SLOPES SHOULD BE A MINIMUM 1V:6H WITH RESPECT TO THE CROSSOVER PAVEMENT, UNLESS PROTECTED BY A GUIDE RAIL.
3. PAVED SURFACE SHALL CONSIST OF: 40 mm HMA WEARING COURSE, 100 mm HMA BASE COURSE, 150 mm SUBBASE OR 40 mm HMA WEARING COURSE, 150 mm AGG-BIT, 150 mm SUBBASE.
4. WHEN CONCRETE SHOULDERS ARE PROVIDED ON THE MAINLINE, THE 1.2 m CONCRETE SHOULDER SHALL BE CONTINUED THROUGH THE DECELERATION LANE AND CROSSOVER AREA. THE ADDITIONAL 2.4 m WIDENING FOR THE DECELERATION LANE AND THE CROSSOVER PAVEMENT SHALL BE A PAVED SURFACE CONSISTING OF: 40 mm HMA WEARING COURSE, 100 mm HMA BASE COURSE, 150 mm SUBBASE OR 40 mm HMA WEARING COURSE, 150 mm AGG-BIT, 150 mm SUBBASE.

FIGURE 2.9 (METRIC)
TYPICAL DUAL MEDIAN
CROSSOVERS AT STATE
OR COUNTY LINES



1. TRANSVERSE SLOPES SHOULD BE A MINIMUM 1V:12H WITH RESPECT TO THE CROSSOVER PAVEMENT, FLAT AND FREE OF ROADSIDE OBSTRUCTIONS.
2. LONGITUDINAL SLOPES SHOULD BE A MINIMUM 1V:6H WITH RESPECT TO THE CROSSOVER PAVEMENT, UNLESS PROTECTED BY A GUIDE RAIL.
3. PAVED SURFACE SHALL CONSIST OF: 1½" HMA WEARING COURSE, 4" HMA BASE COURSE, 6" SUBBASE OR 1½" HMA WEARING COURSE, 6" AGG-BIT, 6" SUBBASE.
4. WHEN CONCRETE SHOULDERS ARE PROVIDED ON THE MAINLINE, THE 4' CONCRETE SHOULDER SHALL BE CONTINUED THROUGH THE DECELERATION LANE AND CROSSOVER AREA. THE ADDITIONAL 8' WIDENING FOR THE DECELERATION LANE AND THE CROSSOVER PAVEMENT SHALL BE A PAVED SURFACE CONSISTING OF: 1½" HMA WEARING COURSE, 4" HMA BASE COURSE, 6" SUBBASE OR 1½" HMA WEARING COURSE, 6" AGG-BIT, 6" SUBBASE.

FIGURE 2.9 (ENGLISH)
TYPICAL DUAL MEDIAN
CROSSOVERS AT STATE
OR COUNTY LINES

2.19 DESIGN CONTROLS

This section presents the characteristics of design vehicles, driver performance and traffic data that are necessary for the optimization or improvement in the design of the various highways that comprise the functional classification system. Additional sources of information and criteria to supplement the general characteristics presented are contained in the 2004 AASHTO Green Book, Chapter 2, "Design Controls and Criteria".

A. Design Vehicles. Design vehicles represent selected motor vehicles with the weight, dimensions and operating characteristics used to establish highway design controls for accommodating vehicles of designated classes. Nineteen design vehicles are used in design that comprise four general classes, including passenger cars, buses, trucks and recreational vehicles. In the design of any highway facility, the largest design vehicle likely to use that facility with considerable frequency or a design vehicle with special characteristics appropriate to a particular intersection is used to determine the design of such critical features as radii at intersections and radii of turning roadways. The dimensions for the 19 design vehicles and their associated symbols are presented in the 2004 AASHTO Green Book, Chapter 2, Exhibit 2-1.

1. **Minimum Turning Paths.** The minimum turning paths for the 19 design vehicles are presented in the 2004 AASHTO Green Book, Chapter 2, Exhibits 2-3 through 2-23. The boundaries of the minimum turning paths are established by the outer trace of the front overhang and the path of the inner rear wheel. The 2004 AASHTO Green Book, Chapter 2, Exhibit 2-2 indicates the minimum turning radius and the minimum inside radius for the 19 design vehicles.
2. **Vehicle Performance.** Acceleration and deceleration rates of vehicles represent critical parameters in determining highway design and these rates often govern the dimensions of such design features as intersections, freeway ramps, climbing or passing lanes and turnout bays for buses.
3. **Vehicular Pollution.** Pollutants emitted from motor vehicles and pollutants in the form of noise transmitted to the surrounding area are factors that shall be recognized during the highway design process. Factors including vehicle mix, vehicle speed, ambient air temperature, vehicle age distribution and the percentage of vehicles operating in a cold mode affect the rate of pollutant emission from vehicles. For passenger cars, noise produced under normal operating conditions is primarily from the engine exhaust system and the tire-roadway interaction. Truck noise has several principal components originating from such sources as exhaust, engine gears, fans and air intake.

The quality of noise varies with the number and operating conditions of the vehicles while the directionality and amplitude of the noise vary with highway design features. The highway designer shall therefore be concerned with how highway location and design influence the vehicle noise perceived by persons residing or working nearby. The perceived noise level decreases as the distance to the highway from a residence or workplace increases.

B. Driver Performance. An appreciation of driver performance is essential to proper highway design and operation. When drivers use a highway designed to be compatible with their capabilities and limitations, their performance is aided. Where positive guidance is applied to design, competent drivers, using well-designed highways with appropriate information displays, can perform safely and efficiently.

The section "Driver Performance" in the 2004 AASHTO Green Book, Chapter 2 provides additional information that is useful in designing and operating highways. It describes drivers in terms of their performance---how they interact with the highway and its information system and why they make errors. Specifically, this section discusses:

1. Older Drivers.
2. The Driving Task.
3. The Guidance Task.
4. The Information System.
5. Information Handling.
6. Driver Error.
7. Speed and Design.
8. Design Assessment.

C. Traffic Characteristics. The design of a highway and its features should be based upon explicit consideration of the traffic volume information which serves to establish the loads for the geometric highway design. The data collected include traffic volumes for days of the year and times of the day, the distribution of vehicles by types and weights and information on trends from which the designer may estimate the traffic expected in the future.

The section "Traffic Characteristics" in the 2004 AASHTO Green Book, Chapter 2 provides additional information about the following:

1. Traffic Volumes.

a. Average Daily Traffic (ADT) Volume. Defined as the total volume during a given time period (in whole days), greater than one day and less than one year, divided by the number of days in that time period.

b. Hourly Traffic Volume. Knowledge of the ADT volume is important for many purposes; however, the direct use of ADT volume in the geometric design of highways is not appropriate since it does not indicate traffic volume variations occurring during the various months of the year, days of the week and hours of the day. Traffic volumes for an interval of time shorter than a day more appropriately reflect operating conditions to be used for design. The hourly traffic volume used in design should not be exceeded very often or by very much nor should it be so high that traffic would rarely be sufficient to make full use of the resulting facility. One guide to determine the hourly traffic volume that is best suited for use in design is a curve showing variation in hourly traffic volumes during the year as indicated in the 2004 AASHTO Green Book, Chapter 2, Exhibit 2-28. The hourly traffic best suited for use in design is the 30th highest hourly volume of the year (30 HV). The design hourly volume (DHV), therefore, should be 30 HV of the future year chosen for design. In rural areas with average fluctuation in traffic flow, 30 HV is approximately 15 percent of the ADT while for urban areas 30 HV is approximately 10 percent of the ADT. For the design of a highway improvement, the variation in hourly traffic volumes should be measured and the percentage of ADT during the 30th highest hour determined. Where such measurement cannot be made and the ADT only is known, use should be made of 30 HV percentage factors for similar highways in the same locality, operated under similar conditions.

2. Directional Distribution. The directional distribution of traffic on multilane facilities during the design hour (DDHV) may be computed by multiplying the ADT by the percentage that 30 HV is of the ADT and then by the percentage of traffic in the peak direction during the design hour.

3. Composition of Traffic. Truck traffic should be expressed as a percentage of total traffic during the design hour (in the case of a two-lane highway, as a percentage of total two-way traffic, and in the case of a multilane highway, as a percentage of total traffic in the peak direction of travel).

4. Projection of Future Traffic Demands. New highways or improvements to existing highways should not usually be based on current traffic volumes alone, but should consider future traffic volumes expected to use the facility. A period of 20 years should be used as the basis for design. For reconstruction or rehabilitation projects, estimating traffic volumes for a 20-year design period may not be appropriate because of the uncertainties of predicting traffic and funding constraints. A shorter design period (5 to 10 years) may be developed for such projects.

5. Speed.

a. Operating Speed. Operating speed is the speed at which drivers are observed operating their vehicles during free-flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or geometric feature.

b. Design Speed. Design speed is a selected speed used to determine the various geometric design features of the roadway.

c. **Running Speed.** The speed at which an individual vehicle travels over a highway section, defined as the length of the highway section divided by the running time required for the vehicle to travel through the section.

6. **Traffic Flow Relationships.** Traffic flow conditions on roadways can be characterized by the volume flow rate expressed in vehicles per hour, the average speed in kilometers per hour (miles per hour) and the traffic density in vehicles per kilometer (vehicles per mile). Generalized speed-volume-density relationships are shown in the 2004 AASHTO Green Book, Chapter 2, Exhibit 2-30.

D. Safety. The section "Safety" in the 2004 AASHTO Green Book, Chapter 2 discusses how a viable safety evaluation and improvement program is a vital part of the overall highway improvement program. Areas of primary importance include the identification of potential safety problems, the evaluation of the effectiveness of alternative solutions, and the programming of available funds for the most effective improvements.

E. Environment. The section "Environment" in the 2004 AASHTO Green Book, Chapter 2 discusses how a highway should be considered as an element of the total environment. Because highway location and design decisions have an effect on the development of adjacent areas, it is important that environmental variables be given full consideration. Also, care should be exercised to ensure that applicable local, state, and federal environmental requirements are met.

F. Economic Analysis. Highway economics is concerned with the cost of a proposed improvement and the benefits resulting from it. The AASHTO publication, "User Benefit Analysis for Highways", may be used to perform economic analysis of proposed highway improvements.

2.20 VERTICAL CLEARANCE REQUIREMENTS

Vertical clearance represents one of the key highway elements or features as the controlling criteria for developing geometric design for both highway and bridge projects.

As such, the clearances presented in this Section represent the minimum acceptable criteria and shall be used as the required vertical control based on the functional classification of the facility and type of project. Vertical clearance shall apply to the required clearance over the entire roadway width and the usable width of the shoulders and shall also include auxiliary lanes, when applicable, to structures passing over the highway facility. The minimum vertical clearance required shall preferably be maintained within the recovery area. See [Table 2.1](#) for a summary of required vertical clearances over roadways.

All structures having a vertical clearance below the minimum acceptable criteria should ultimately be considered for improvement of clearance. When the vertical clearance requirements cannot be achieved, justification to support a design exception submission request shall be provided.

A. Strategic Highway Network (STRAHNET). The Surface Deployment and Distribution Command Transportation Engineering Agency (SDDCTEA) of the Department of Defense has developed and continues to refine the Strategic Highway Network (STRAHNET). The STRAHNET is a system of highways that provides defense access, continuity and emergency capabilities for movements of personnel and equipment in both peacetime and wartime. STRAHNET routes are included on the National Highway System. STRAHNET routes include all interstate highways in Pennsylvania (including the Pennsylvania Turnpike interstate highways), strategic highway network routes and major strategic highway network connectors. For a map of the STRAHNET see: ftp://ftp.dot.state.pa.us/public/pdf/BPR_PDF_FILES/MAPS/Statewide/STRAHNET_web_map.pdf.

All highway facilities on the STRAHNET require the vertical clearances as noted on [Table 2.1](#).

When a vertical clearance of less than 4.9 m (16 ft, 0 in) is created as a result of a highway construction project on the STRAHNET, it is considered an exception. All exceptions to the 4.9 m (16 ft, 0 in) vertical clearance standard on rural Interstate routes or on a single Interstate route through urban areas require coordination with the SDDCTEA. Coordination should occur whether it is a new construction project, a project that does not provide for correction of an existing substandard condition, or a project which creates a substandard vertical clearance. This

applies to the full roadway width including shoulders for the through lanes, as well as ramps and collector-distributor roadways for Interstate-to-Interstate interchanges.

A request for coordination may be forwarded by FHWA to the SDDCTEA at any time during project development prior to taking any action on the design exception. It should include a time period of 10 working days (after receipt) for action on the request. The FHWA office initiating a request for coordination to the SDDCTEA should verify receipt of the request by telephone or fax. If the SDDCTEA does not respond within the time frame, the FHWA should conclude that the SDDCTEA does not have any concerns with the proposed exception. If comments are forthcoming, the FHWA and the Department will consider mitigation to the extent feasible.

Chapter 2, Appendix A provides a form and instructions that should be used when requesting vertical clearance design exception coordination with the SDDCTEA. FHWA submits this form to the SDDCTEA.

B. Bridges over Railroads. The vertical clearance requirements for all bridges over railroads shall conform with the criteria presented in Publication 15M, Design Manual, Part 4, *Structures*, Section D2.3.3.4 and Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures*, Section 4.11.D.

C. Pedestrian Overpasses.

1. New Construction, Reconstruction and Superstructure Replacement Projects. Because of their lesser resistance to impacts, the vertical clearance requirements for all pedestrian overpass structures shall be 0.30 m (1 ft) greater than the vertical clearance required for the highway over which the structure is located.
2. 3R Projects. Because of their lesser resistance to impacts, the vertical clearance requirements for all pedestrian overpass structures shall be 0.30 m (1 ft) greater than the 3R vertical clearance required for the highway over which the structure is located. Pedestrian overpasses over arterials may remain in place for vertical clearance down to 4.6 m (15 ft, 0 in), but the vertical clearance may not be further reduced.
3. Pavement Preservation Projects. Because of their lesser resistance to impacts, the vertical clearance requirements for all pedestrian overpass structures shall be at least 0.30 m (1 ft) greater than the 3R vertical clearance required for the highway over which the structure is located. The vertical clearances presently below minimum requirements shall not be further reduced by a Pavement Preservation project as per Pavement Preservation Guidelines presented in Publication 242, *Pavement Policy Manual*, Appendix G.
4. Bridge Preservation Projects. Because of their lesser resistance to impacts, the vertical clearance requirements for all pedestrian overpass structures shall be at least 0.30 m (1 ft) greater than the 3R vertical clearance required for the highway over which the structure is located. The vertical clearances presently below minimum requirements shall not be further reduced by a Bridge Preservation project. Refer to Publication 15M, Design Manual, Part 4, *Structures*, for work eligible for Bridge Preservation projects.

D. Traffic Signals. The vertical clearance requirements shall conform with the criteria presented in Publication 149, *Traffic Signal Design Handbook*.

E. Utility Lines. The vertical clearance requirements shall conform with the criteria presented in Publication 16M, Design Manual, Part 5, *Utility Relocation*, Appendix A.

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TABLE 2.1 (METRIC)
REQUIRED VERTICAL CLEARANCES FOR STRUCTURES OVER HIGHWAYS

Type of Project ⁽¹⁾	STRAHNET	Freeways	Arterials	Collectors and Local Roads	Overhead Sign Structures ⁽²⁾
New Construction, Reconstruction & Superstructure Replacements	5.05 m	5.05 m	5.05 m	4.45 m	5.33 m
3R	4.9 m ⁽³⁾	Not Applicable	4.9 m ⁽⁵⁾	4.3 m	5.18 m
Deck Replacement, Pavement Preservation & Bridge Preservation ⁽⁴⁾	4.9 m	4.9 m	4.9 m	4.3 m	5.18 m

- (1) For vertical clearance under pedestrian bridges, see [Section 2.20.C](#).
- (2) Details regarding the vertical clearance requirements are presented in Publication 218M, *Standards for Bridge Design*, BD-600 (Dual Unit).
- (3) 3R criteria is not applicable for freeways.
- (4) Existing vertical clearances below minimum requirements shall not be further reduced by a Deck Replacement, a Pavement Preservation or a Bridge Preservation project. Refer to Publication 15M, Design Manual, Part 4, *Structures*, for work eligible for Bridge Preservation projects. For Pavement Preservation Guidelines, see Publication 242, *Pavement Policy Manual*, Appendix G.
- (5) Existing vertical clearances below 4.9 m, but over 4.3 m can remain for arterials, but are not to be further reduced.

TABLE 2.1 (ENGLISH)
REQUIRED VERTICAL CLEARANCES FOR STRUCTURES OVER HIGHWAYS

Type of Project ⁽¹⁾	STRAHNET	Freeways	Arterials	Collectors and Local Roads	Overhead Sign Structures ⁽²⁾
New Construction, Reconstruction & Superstructure Replacements	16'-6"	16'-6"	16'-6"	14'-6"	17'-6"
3R	16'-0" ⁽³⁾	Not Applicable	16'-0" ⁽⁵⁾	14'-0"	17'-0"
Deck Replacement, Pavement Preservation & Bridge Preservation ⁽⁴⁾	16'-0"	16'-0"	16'-0"	14'-0"	17'-0"

- (1) For vertical clearance under pedestrian bridges, see [Section 2.20.C](#).
- (2) Details regarding the vertical clearance requirements are presented in Publication 218M, *Standards for Bridge Design*, BD-600 (Dual Unit).
- (3) 3R criteria is not applicable for freeways.
- (4) Existing vertical clearances below minimum requirements shall not be further reduced by a Deck Replacement, a Pavement Preservation or a Bridge Preservation project. Refer to Publication 15M, Design Manual, Part 4, *Structures*, for work eligible for Bridge Preservation projects. For Pavement Preservation Guidelines, see Publication 242, *Pavement Policy Manual*, Appendix G.
- (5) Existing vertical clearances below 16'-0", but over 14'-0" can remain for arterials, but are not to be further reduced.