



Chapter 6 Elements of Design

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Chapter

6



6 Elements of Design

6.1 Introduction

The design of Roads; Streets; and Freeways, Expressways, and Interstates in differing context classifications is treated differently. Information on these specific facility types can be found in Chapters 3, 4, and 5 of this Guide, respectively. However, common to all facility types are several principal elements of design—sight distance, cross slope, travel lane width, and horizontal and vertical alignments. These elements are discussed in this chapter, and, as appropriate, in Chapters 3, 4, and 5 pertaining to those specific facility types.

The alignment of facilities has the potential to increase or mitigate impacts on the environment, community, and roadway user. The alignment is comprised of a variety of elements joined together to create a facility that serves the roadway user in a safe and efficient manner, consistent with the facility's intended function. Each alignment element should complement the others to produce a consistent, safe, context-sensitive, efficient, and environmentally responsible design.

6.2 Sight Distance

6.2.1 General Considerations

A critical element in assuring safe and efficient operation of vehicles on a roadway is the ability of the driver to see ahead. Sight distance is the distance along a roadway throughout which an object of specified height is continuously visible to the driver. Sight distance requirements can vary depending upon the intended actions of the driver and vehicle. Sight distance will vary if the driver is departing the highway, entering the highway, or attempting to pass a vehicle. These are just a few examples of the differing actions and sight distance requirements that need to be considered by the designer. This distance is also 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 such as cut slopes, guardrail, and retaining walls within the driver's line of sight. Sight distance of sufficient length must be provided to allow drivers to avoid striking unexpected objects in the traveled way. Certain two-lane roadways should also provide sufficient sight distance to allow drivers to occupy the opposing lane for passing without hazard.



When designing a separated shared-use path, the design vehicle will likely be a bicycle and have different sight distance requirements for the path design. Refer to Chapter 13 of this Guide for more information on bicycle design and Chapter 12 of this Guide for accessibility guidelines.

6.2.2 Stopping Sight Distance for Roadways

Stopping sight distance along a roadway is the sum of two distances (reaction distance and braking distance). This calculation and further details regarding stopping sight distance can be found in Chapter 3, Section 3.2.2.3, of the American Association of State Highway and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets* (the Green Book) (2018 AASHTO GDHS) (AASHTO, 2018).

- **Reaction Distance.** The distance a vehicle travels from the instant the driver sights an object necessitating a stop to the instant the brakes are applied (brake reaction distance), and
- **Braking Distance.** The distance required to stop the vehicle from the instant brake application begins.

$$SSD = 1.47Vt + 1.075 \frac{V^2}{a}$$

SSD= stopping sight distance, ft
 V = design speed, mph
 t = brake reaction time, 2.5s
 a = deceleration rate, $\frac{ft}{s^2}$

Stopping sight distance is measured from the passenger vehicles driver's eyes, which are assumed to be 3.5 feet above the pavement, to an object 2 feet high on the road. Distances greater than the minimum stopping sight distance provide an additional measure of safety and should be considered where practical. For locations with increased truck activity, it may be appropriate to use different operator heights to calculate sight distance depending upon the vehicle type. Refer to Chapter 3, Section 3.2.2.5, of the 2018 AASHTO GDHS for additional information.

Table 6-1 is referred to in this chapter for sight distance adjustments.



Table 6-1 Sight Distance Adjustments for Grades or Vertical Curves

| Design Speed (mph) | Stopping Sight Distance (Design Values) | | | | | | | | | Passing Sight Distance (2-Lane Road) | | |
|--------------------|---|--------------|------|------|------------|-----|-----|--------|--------|--------------------------------------|--------|--|
| | No Grade Adjustment | % Down Grade | | | % Up Grade | | | Crest | Sag | Crest Vertical Curve | | |
| | Dist. (ft) | 3 | 6 | 9 | 3 | 6 | 9 | K | K | Dist. (ft) | K | |
| 15 | 80 | 80 | 82 | 85 | 75 | 74 | 73 | 3 | 10 | 400 | 57 | |
| 20 | 115 | 116 | 120 | 126 | 109 | 107 | 104 | 7 | 17 | | | |
| 25 | 155 | 158 | 165 | 173 | 147 | 143 | 140 | 12 | 26 | 450 | 72 | |
| 30 | 200 | 205 | 215 | 227 | 200 | 184 | 179 | 19 | 37 | 500 | 89 | |
| 35 | 250 | 257 | 271 | 287 | 237 | 229 | 222 | 29 | 49 | 550 | 108 | |
| 40 | 305 | 315 | 333 | 354 | 289 | 278 | 269 | 44 | 64 | 600 | 129 | |
| 45 | 360 | 378 | 400 | 427 | 344 | 331 | 320 | 61 | 79 | 700 | 175 | |
| 50 | 425 | 446 | 474 | 507 | 405 | 388 | 375 | 84 | 96 | 800 | 229 | |
| 55 | 495 | 520 | 553 | 593 | 469 | 450 | 433 | 114 | 115 | 900 | 289 | |
| 60 | 570 | 598 | 638 | 686 | 538 | 515 | 495 | 151 | 136 | 1000 | 357 | |
| 65 | 645 | 682 | 728 | 785 | 612 | 584 | 561 | 193 | 157 | 1100 | 432 | |
| 70 | 730 | 771 | 825 | 891 | 690 | 658 | 631 | 247 | 181 | 1200 | 514 | |
| 75 | 820 | 866 | 927 | 1003 | 772 | 736 | 704 | 312 | 206 | 1300 | 604 | |
| 80 | 910 | 965 | 1035 | 1121 | 859 | 817 | 782 | 384 | 231 | 1400 | 700 | |
| AASHTO Tables | (3-1) | (3-2) | | | | | | (3-35) | (3-37) | (3-5) (3-36) | (3-36) | |

Source: 2018 AASHTO GDHS.

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

Where:

L = Length of vertical curve (ft)

A = Algebraic difference in intersecting grades, in percent

K value is a coefficient by which the algebraic difference in grade is multiplied to determine the length in feet of the vertical curve that will provide minimum sight distance. Values of K=167 or greater should be checked for drainage.

Refer to Chapter 3, Section 3.2.2, of the 2018 AASHTO GDHS for more information.

6.2.2.1 Effect of Grade on Stopping Sight Distance

Stopping distances on upgrades are shorter than on flat terrain and conversely longer in distance on downgrades. Design speed is used in calculating downgrade corrections; average running speed is used in calculating upgrade corrections. The different criteria for descending and ascending grades are based on the effect grades have on the speed of individual vehicles, particularly trucks; the effect these vehicles have on the overall speed of the traffic stream; and the premise that many drivers, particularly those in automobiles, do not appropriately compensate for the changes in speed caused by grades.

Most facilities support two-way traffic, the roadway grade is traversed by traffic in both directions, but the sight distance at any point on the roadway generally is different for each direction, particularly on straight roads in rolling terrain. As a general rule, the sight distance available on downgrades tends to be greater than on upgrades, more or less automatically providing the necessary corrections for grade. Exceptions are one-way roadways or divided roadways with independent design profiles for the two roadways. In these locations separate grade corrections are necessary and the refinement in design is consistent with the overall standards used.

For those areas where there is a high volume of trucks, refer to Chapter 3, Section 3.2.2.5, of the 2018 AASHTO GDHS for more information on adjustments for large vehicles.

6.2.3 Decision Sight Distance

Stopping sight distance may not always be adequate when drivers are required to make complex or instantaneous decisions, when information is difficult to perceive, or when unexpected or unusual maneuvers are required. In these instances, stopping sight distances may not provide sufficient visibility distance for drivers to corroborate advance warnings and to perform the necessary maneuvers. Decision sight distance provides the greater length that drivers need in these instances.

The 2018 AASHTO GDHS provides values for appropriate decision sight distances at critical locations and for criteria in evaluating the suitability of the sight lengths at these locations. Refer to Chapter 3, Section 3.2.3, of the 2018 AASHTO GDHS for more information.

Decision sight distance is the distance required for a driver to:

- Detect an unexpected or otherwise difficult to perceive information source or hazard in a roadway environment that may be visually cluttered.
- Recognize the hazard or its threat potential.
- Select an appropriate speed and path.
- Initiate and complete the required safety maneuver safely and efficiently.

Drivers need decision sight distances whenever there is a likelihood for error in either information reception, decision making, or control actions. 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:

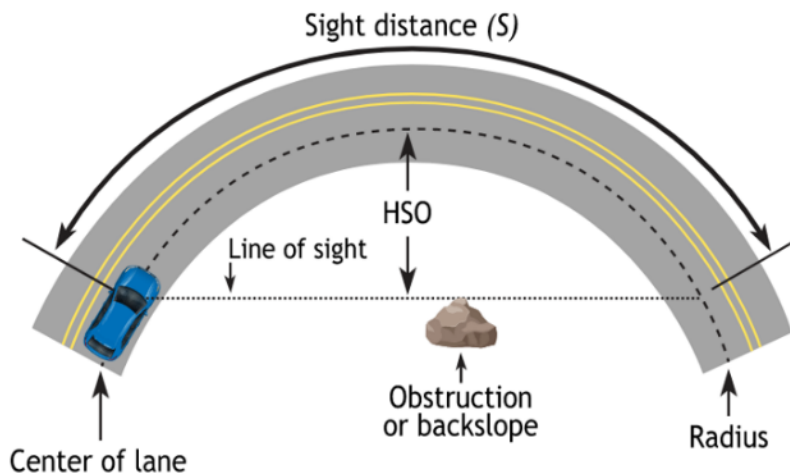
- Interchange and intersection locations where unusual or unexpected maneuvers are required.
- Changes in cross section such as tunnels and lane drops.
- Areas of concentrated demand where there is apt to be "visual noise" whenever sources of information compete, as those from roadway elements, traffic, traffic control devices, surrounding land use, pedestrian activity, and advertising signs.

6.2.4 Sight Distance on Horizontal Curves

Stopping sight distance on horizontal curves may be obtained with the aid of Figures 6-1. For passenger vehicles, it is assumed that the driver's eyes are 3.5 feet above the center of the inside lane (inside with respect to the curve) and the object is 2 feet high. The line of sight is assumed to intercept the obstruction at the midpoint of the sight line and 2 feet above the center of the inside lane. The middle horizontal sightline offset (HSO) is obtained from Figure 6-1.

Sight distance obstructions should be considered in the design; refer to Chapter 3, Section 3.3.12.1, of the 2018 AASHTO GDHS.

Figure 6-1 Horizontal Stopping Sight Distance



$$S = \frac{R_i}{28.65} \arccos\left(\frac{R_i - HSO}{R_i}\right)$$

Where:

$R_i = R$ (Radius of curve) \pm Offset to center of driving lane (ft)

HSO = Horizontal sightline offset distance from center of inside lane to object or obstruction (ft)

S = Available stopping distance (ft)



Example PBDP Application - Calculating Sight Distance

When calculating sight distance, the designer shall always calculate the radius to the center of the lane with the most limited sight distance. The horizontal sightline offset shall also be calculated to the center of the lane.

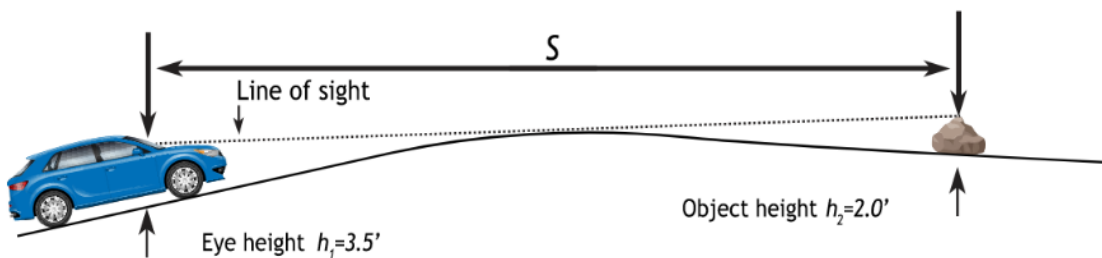
On a divided highway with a median barrier, the shortest sight distance could be the closest inside lane to the obstruction (cliff or hillside) relatively close to the lane, or it could be the inside lane closest to the median barrier. The designer should check both conditions to determine the most restrictive case to design to. There are software design tools to help the designer check sight distance, but the designer must check the settings in the software to make sure the vehicle and object heights are set appropriately.

6.2.5 Sight Distance on Vertical Curves

6.2.5.1 Crest Vertical Curves

Stopping sight distance is measured when the height of eye is 3.5 feet, and the height of object is 2 feet, as shown in Figure 6-2.

Figure 6-2 Stopping Sight Distance on a Crest Vertical Curve



When: $S < L$ $L = \frac{AS^2}{2158}$ $S = \sqrt{\frac{2158L}{A}}$

When: $S > L$ $L = 2S - \frac{2158}{A}$ $S = \frac{L}{2} + \frac{1079}{A}$

Where:

- L = Length of sag vertical curve (ft)
- A = Algebraic difference in grades (%)
- S = Light beam distance (ft)

6.2.5.2 Sag Vertical Curves

Headlight sight distance is the basis for determining the length of sight distance. Prior to calculating the following formula, review Table 3-37 of the 2018 AASHTO GDHS to ascertain if S is less than or greater than L.

When: $S < L$ $S = \frac{3.5L \pm \sqrt{12.25L^2 + 1600AL}}{2A}$



When: $S > L$ $S = \frac{AL+400}{2A-3.5}$

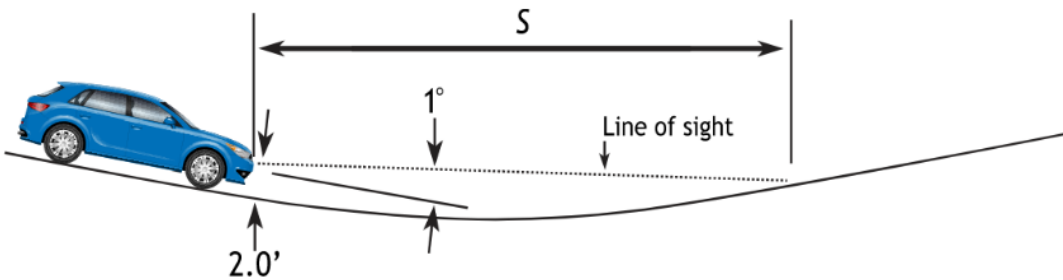
Where:

L = Length of sag vertical curve (ft)

A = Algebraic difference in grades (%)

S = Light beam distance (ft)

Figure 6-3 Example of a Sag Vertical Curve



Given:

L = 300 feet, check if curve is adequate for a design speed of 40 MPH, and find S.

$$A = 2.5 - (-2.0) = 4.5$$

$$K = \frac{L}{A} = \frac{300}{4.5} = 66.7$$

From "Sag K" column of Table 6-1, with K(Sag) value of 64 for 40 mph. Curve is adequate for a design speed of 40 mph.

Since it is unknown whether $S > L$ or $S < L$, try each equation or consult Figure 3-37 Design Controls for Sag Vertical Curves in the 2018 AASHTO GDHS.

Try:

$$S < L \rightarrow S = \frac{3.5L \pm \sqrt{12.25L^2 + 1600AL}}{2A} = \frac{3.5(300) \pm \sqrt{12.25(300)^2 + 1600(4.5)(300)}}{2(4.5)}$$

$$S = 317 \text{ ft. , which is greater than } L = 300 \text{ ft.}$$

which is not good, S is not less than L

Try:

$$S > L \rightarrow S = \frac{AL+400}{2A-3.5} = \frac{(4.5)(300)+400}{2(4.5)-3.5} = 318 \text{ ft.}$$

Which is okay, S is greater than L



6.2.5.3 Passing Sight Distance

Passing sight distance is the minimum sight distance required for the driver of one vehicle to pass another vehicle safely. Passing sight distance is considered only on two-lane roadways. Passing sight distance is measured between an eye height of 3.5 feet and an object height of 3.5 feet. Table 6-1 presents minimum passing sight distances for various design speeds. Refer to Section 3.4.6.2.2, of the 2018 AASHTO GDHS for more information.

Generally, it is impractical to design crest vertical curves to provide for passing sight distance because of the high cost where crest cuts are involved and the difficulty of fitting the required long vertical curves to the terrain, particularly for high-speed roads.

Passing sight distance calculations are for design purposes only to assist in providing as many passing opportunities as possible. Actual passing and no-passing zone locations for striping need to be field measured and placed in accordance with the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) (FHWA, [2009] 2022).



When vehicles are passing a bicyclist, there needs to be room for the bicyclist both for comfort and safety. In accordance with Colorado State law, a driver must maintain a minimum separation of 3 feet between the vehicle and the bicyclist. The roadway design needs to include the width of the vehicular travel lane and the bicycle lane or shoulder so this distance can be maintained, given the nature of traffic volume in adjacent travel lanes.

6.2.5.4 Passing Sight Distance on Crest Vertical Curves

Design values of crest vertical curves for passing sight distance differ from those for stopping sight distance because of the different height criterion, i.e., 3.5 feet for the height of object for passing sight distance compared to 2 feet for stopping sight distance. The following formulas apply:

$$\text{When } S > L \quad L = 2S - \frac{2800}{A} \quad S = \frac{L}{2} + \frac{1400}{A}$$

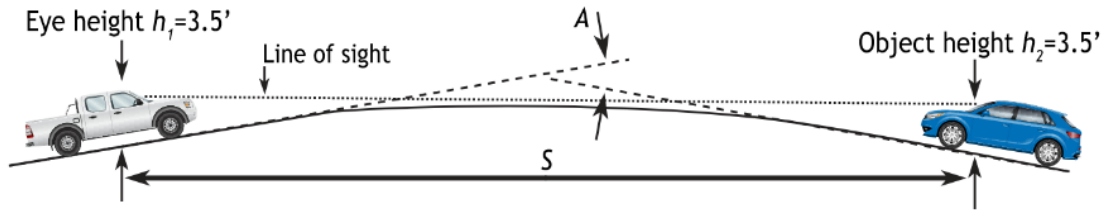
$$\text{When } S < L \quad L = \frac{AS^2}{2800} \quad S = \sqrt{\frac{2800L}{A}}$$

Where:

- L = Length of vertical curve (ft)
- A = Algebraic grade difference (%)
- S = Sight Distance (ft)

For minimum passing sight distances, the required lengths of crest vertical curves are substantially longer than those for stopping sight distances, as evidenced by the values in Table 6-1.

Figure 6-4 Example of Passing Sight Distance on Crest Vertical Curve



6.2.5.5 Passing and Stopping Sight Distances at Undercrossings

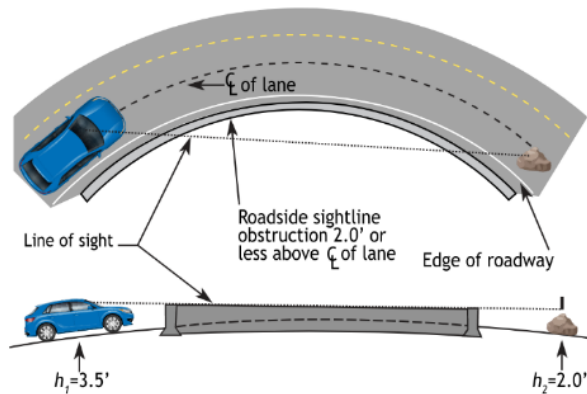
If economically feasible, passing sight distance should be maintained as the roadway passes under a structure. On occasion, topographic conditions may result in a pronounced sag curve, and the underside of the structure may limit the sight distance. Such conditions may best be checked graphically on the profile using the vertical clear dimension of the structure, the height of the eye for a truck driver as 7.6 feet and the height of object as 2 feet for the taillights of a vehicle.

Refer to Chapter 3, Section 3.2.4, of the 2018 AASHTO GDHS for more information.

6.2.5.6 Overlapping Horizontal and Vertical Curves

Vertical curves on a horizontal curve have a unique effect that the designer should consider very carefully. Crest vertical curves make roadside objects more likely to hinder the ability of the driver to see the obstruction ahead, as shown in Figure 6-5. Sag vertical curves may make roadside objects less likely to be a sight line obstruction.

Figure 6-5 Overlapping Horizontal and Vertical Curves



The designer may need to check sight lines graphically with software to determine if the roadside object is hindering the calculated passing and/or stopping sight distance requirements on a roadway with both a horizontal and vertical curve.

6.3 Horizontal Alignment

6.3.1 Theoretical Considerations

As stated in the 2018 AASHTO GDHS,



“To achieve balance in highway design, all geometric elements should, as far as economically practical, be designed to operate at a speed likely to be observed under the normal conditions for that roadway for a vast majority of motorists.”

Refer to Chapter 3, Section 3.3, of the 2018 AASHTO GDHS for more information.

6.3.2 General Considerations

Horizontal alignment should provide for safe and continuous operation of vehicles at a uniform design speed for substantial lengths of roadway. The major considerations in horizontal alignment are:

- Topography.
- Type of facility.
- Design speed.
- Design vehicle.
- Traffic volumes and vehicle classification.
- Profile grade.
- Subsurface conditions.
- Existing roadway and development.
- Existing and future land use.
- Location of the roadway terminals.
- Right of way.
- Safety.
- Construction costs.
- Environmental and seasonal weather conditions.
- Ecological and geological resources.
- Drainage.

All the above considerations should be balanced to produce an alignment that is contextually appropriate for the location and functional classification of the roadway. Functional classification is explained in Chapter 1 of this Guide.

To a large extent, topography controls both curve radius and design speed. In mountainous areas or areas subject to icing, consideration should be given to locating the road so that it has a southern exposure wherever possible.

Geological features that may affect design, such as potential land and rock slide areas along with subsurface water, should be investigated by the CDOT Materials and Geotechnical Branch.

Sight distance, compatible with the selected design speed, is required for proper design.

Horizontal alignment must afford at least the minimum stopping sight distance for the design speed at all points on the roadway, as given in Table 6-1.

Every effort should be made to exceed the minimum curve radii. Minimum curve radii should be used only when the cost of realizing a higher standard is not consistent with the benefits. The final considerations for the safety of any curve should be the combination of the factors of radius, sight distance, curve length, and superelevation.

To avoid the appearance of inconsistent distribution, the horizontal alignment should be coordinated carefully with the profile design.

Refer to Chapter 3, Section 3.3, of the 2018 AASHTO GDHS for more information.

6.3.3 MM, PBPD, and CSS Considerations

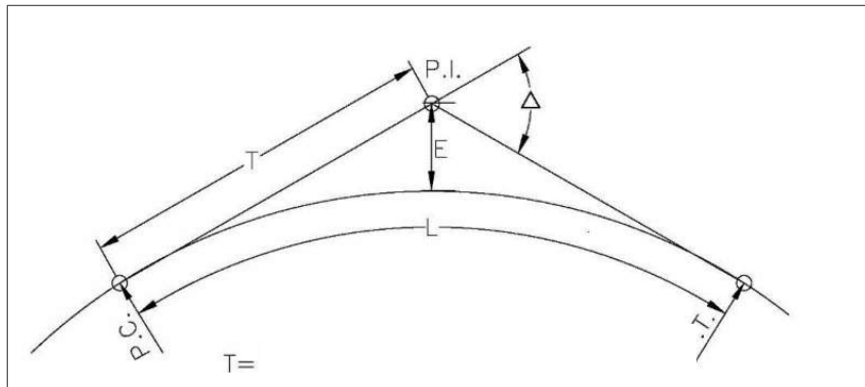
Development of horizontal alignments need to account for all modes of transportation based on the purpose and need of the project. The context of the area must be evaluated when determining the project criteria for horizontal alignment. Utilizing performance-based practical design (PBPD) tools and accounting for the aesthetics and character of the area that are important to the stakeholders can help determine the ultimate location of control for the project.

6.3.4 Types and Properties of Horizontal Alignments

6.3.4.1 Simple Curves

A simple curve is a circular arc joining two tangents, as shown in Figure 6-6.

Figure 6-6 Simple Curve



$$T = R \tan \frac{\Delta}{2}$$

$$L = \frac{R \Delta}{5729.58}$$

$$E = R \left(\sec \frac{\Delta}{2} \right) - R = \frac{R}{\cos \frac{\Delta}{2}} - R$$



$$\text{Calculating Number of Chords } P.C.\text{ to }P.T. = \frac{R \Delta}{57.2958}$$

Where:

R = Radius of curve (ft)

L = Length of curve in stations

Δ = Deflection angle between the tangents, decimal degrees

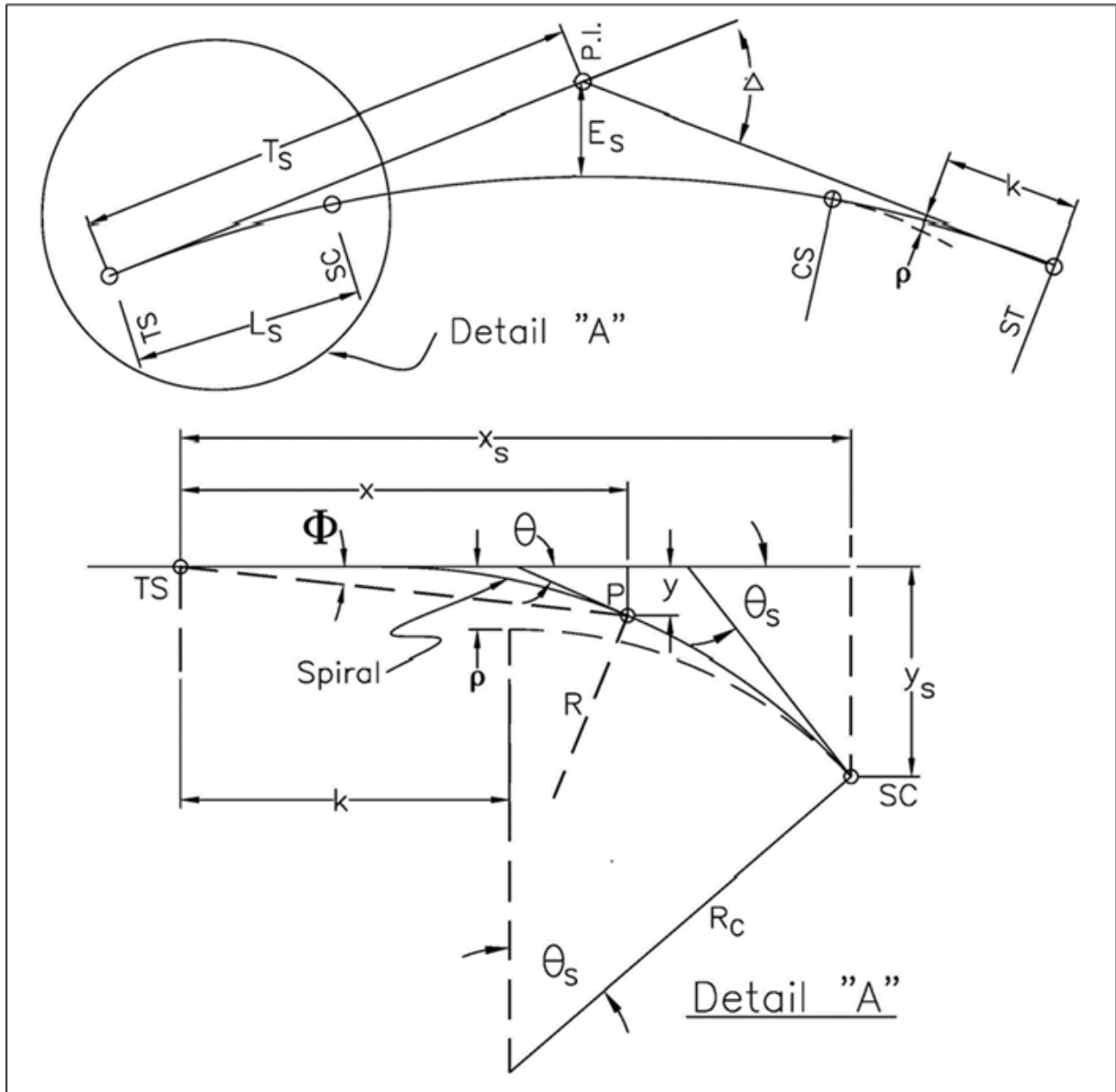
T = Length of tangent (ft)

E = External distance (ft)

6.3.4.2 Spiral Curves

Spiral curves provide a gradual change in curvature from a straight to a circular path. Spiral transitions are not required but may be used on all roadways, including interchange ramps where recommended by the CDOT Standard Plans - M & S Standards on Superelevation (CDOT, 2019) and that include minimum transition lengths to be used with given curvature and speed. For further information on spiral curves, refer to *Route Location and Design* by T .F. Hickerson (Hickerson, 1967).

Figure 6-7 Spiral Curve Detail



P is any point on the spiral curve.



Equations for the spiral curve are as follows:

$$T_s = (R_c + \rho) \tan\left(\frac{\Delta}{2}\right) + k$$

$$E_s = (R_c + \rho) \left(\frac{1}{\cos\left(\frac{\Delta}{2}\right)} - 1 \right) + \rho = \frac{R_c + \rho}{\cos\left(\frac{\Delta}{2}\right)} - R_c$$

$$L_s = \frac{200\theta_s}{D_c}$$

$$\theta = \left(\frac{L}{L_s}\right)^2 \theta_s$$

$$D = \left(\frac{L}{L_s}\right) D_c$$

$$y = \frac{L_s}{100} (0.5818 \theta_s - 0.1266 \times 10^{-4} \theta_s^3)$$

$$x = \frac{L_s}{100} (100 - 0.3046 \times 10^{-2} \theta_s^3)$$

Where:

L is in feet and θ is measured in decimal degrees.

$$\rho = 0.001454\theta_s L_s$$

$$k = L_s (0.5 - 5.0770 \times 10^{-6} \theta_s^2)$$

Where:

θ is measured in decimal degrees.

Where:

TS = point of change from tangent to spiral

SC = point of change from spiral to circle

CS = point of change from circle to spiral

ST = point of change from spiral to tangent

L = spiral arc from the TS to any point on spiral

L_s = total length of spiral from TS to SC

θ = central angle of spiral arc L

θ_s = central angle of spiral arc L_s , called "spiral angle"

Φ = spiral deflection angle at the TS from initial tangent to any point on spiral

D = degree of curve of the spiral at any point

R = radius

D_c = degree of curve of the shifted circle to which the spiral becomes tangent at the SC

R_c = radius of curve of the shifted circle to which the spiral becomes tangent at the SC

Δ = total central angle of the circular curve

Δ_c = central angle of circular arc of length L_c extending from SC to CS

y = tangent offset of any point on spiral with reference to TS and initial tangent

y_s = tangent offset at the SC

x = tangent offset of any point on spiral with reference to TS and initial tangent

x_s = tangent distance for the SC

ρ = offset from the initial tangent to the PC of the shifted circle

k = abscissa of the shifted PC referred to the TS

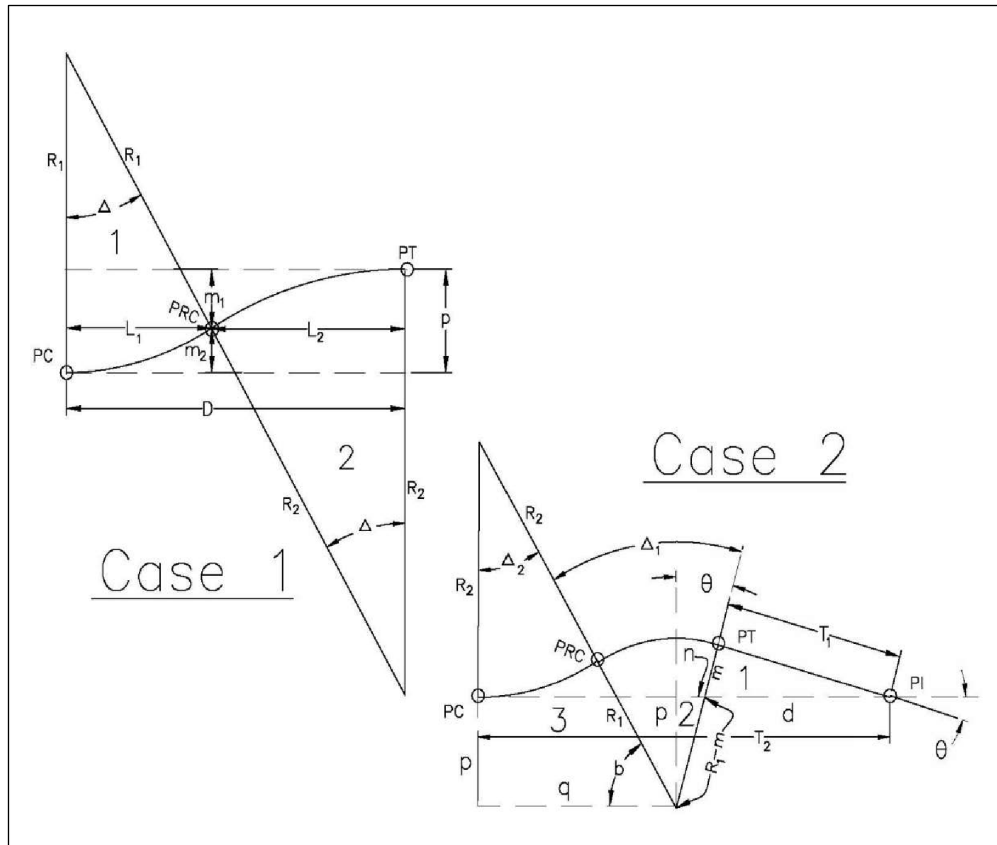
T_s = total tangent distance = distance from PI to TS, or from PI to ST

E_s = total external distance

6.3.4.3 Reverse Curves

Two consecutive circular curves constitute a reverse curve if they join at a point of tangency where their centers are on opposite sides of the common tangent. True reversing of curves should be avoided, although they may at times be used in designing detours. In cases of reversing curves, a sufficient tangent should be maintained between them to avoid overlapping of the required superelevation runoff and tangent runoff.

Figure 6-8 Reverse Curve Case Examples



PRC = Point of Reversing Curvature.

In **Case 1**, the two parallel tangents are to be connected by a reversed curve (such as in a detour).

R_1 , R_2 , and p are given.

From triangle 1,

$$\frac{L_1}{R_1} = \sin \Delta$$

$$L_1 = R_1 \sin \Delta$$

From which,

$$P = m_2 + m_1$$

$$p = R_1(1 - \cos \Delta) + R_2(1 - \cos \Delta)$$

$$p = (R_1 + R_2)(1 - \cos \Delta)$$

And,

$$\frac{R_1 - m_2}{R_1} = \cos \Delta$$

$$\frac{p}{R_1 + R_2} = 1 - \cos \Delta$$

$$m_2 = R_1(1 - \cos \Delta)$$

And,

$$D = L_1 + L_2$$

From triangle 2,

$$\frac{L_2}{R_2} = \sin \Delta$$

$$D = R_1 \sin \Delta + R_2 \sin \Delta$$

$$\frac{R_2 - m_1}{R_2} = \cos \Delta$$

$$D = (R_1 + R_2) \sin \Delta$$

From the preceding equations and the ordinary functions of a simple curve, all ordinary cases of reversed curves between parallel tangents can be solved.

In **Case 2**, the two tangents, intersecting with the angle θ , are to be connected by the reversed curve in which T_1 , R_1 , and R_2 are known, and the tangent distance T_2 and the central angles of the two branches (Δ_1 & Δ_2) are required.

In triangle 1, the base T_1 and the angles are known, from which the sides d and m can be computed.

In triangle 2, the hypotenuse is $R_1 - m$, and the angles are known from which the base p and the altitude n are determined.

In triangle 3, the base is $R_2 + p$, and the hypotenuse is $R_1 + R_2$, from which the angles Δ_2 and b and the distance q can be found.

Then
$$\Delta_1 = \theta + \Delta_2$$

and
$$T_2 = d + n + q$$

6.3.4.4 Compound Curves

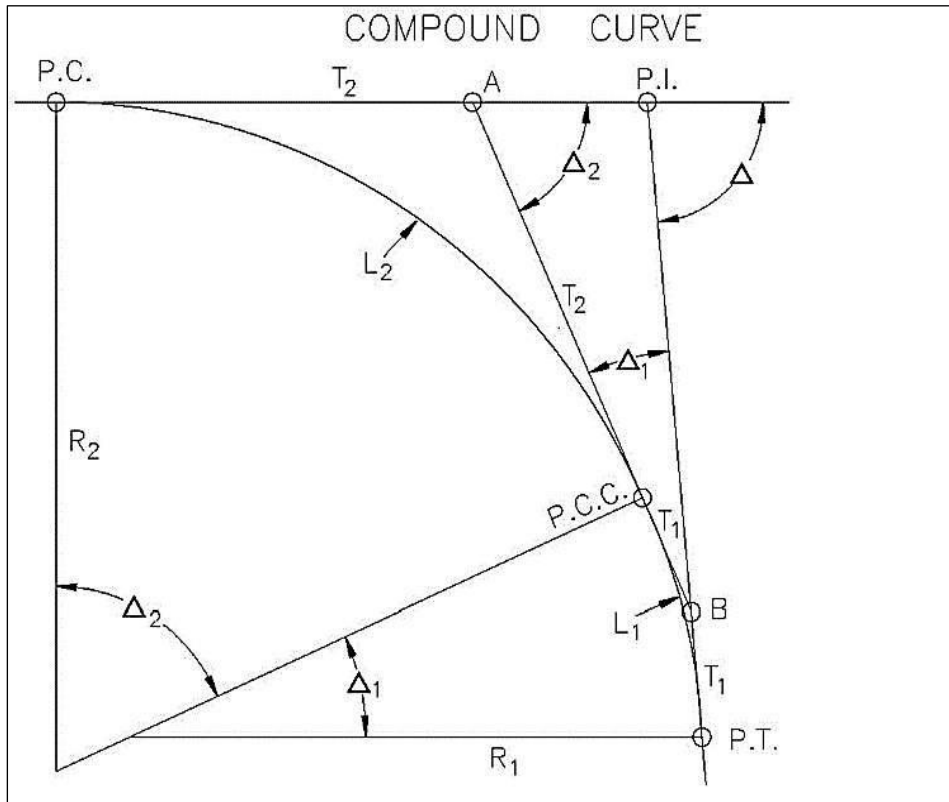
Compound circular curves are two or more consecutive circular curves in the same direction with varying radii. Compound circular curves are joined at a point of tangency and located on the same side of the common tangent.

While simple curves are preferred, compound curves can be used to satisfy topographical constraints that cannot be as effectively balanced with simple curves. For compound curves on open roadways, it is generally accepted that the ratio of the flatter radius to the sharper radius should not exceed 1.5:1. For compound curves at intersections or on ramps, the ratio of the flatter radius to the sharper radius should not exceed 2:1. When this is not feasible, an



intermediate simple curve or spiral should be used to provide the necessary transitions. Since drivers are often unable to distinguish the change in horizontal radii while navigating a compound curve, appropriate warning signs as detailed in the MUTCD should be provided. Refer to the 2018 AASHTO GDHS Chapter 3 for more discussion on compound curves at intersections and Figure 6-9.

Figure 6-9 Compound Curve



Given: $\Delta = \Delta_1 + \Delta_2$ $T_2 = R_2 \tan \frac{\Delta_2}{2}$ $T_1 = R_1 \tan \frac{\Delta_1}{2}$

Curve is laid out as two adjacent simple curves:

- One having a radius “R2” with its P.I. at “A”;
- The other having a radius “R1” with its P.I. at “B”.

On open highways, the ratio R2/R1 should not exceed 1.5:1.
 On other curves (intersections and ramps), the ratio R2/R1 should not exceed 2:1.
 For sharper curves, use a three-centered curve or a spiral.

6.3.4.5 Alignment on Bridges

Ending a curve on a bridge is undesirable and complicates design and construction. Likewise, curves beginning or ending near a bridge should be placed so that no part of the spiral or superelevation transitions extends onto the bridge. Compound curves on a bridge are equally

undesirable. If curvature is unavoidable, every effort should be made to keep the bridge within the limits of the simple curve.

6.3.4.6 Curvature Zoning

In addition to the specific design elements for horizontal alignment discussed previously, a number of general controls are recognized in practice. These controls are not subject to theoretical derivation, but they are important for providing improving the safety and comfort of driver's navigation these highways. Excessive curvature or poor combinations of curvature have the potential to impact traffic operations and decreasing travel speeds, increasing operating and maintenance costs and reduces the overall driver comfort. To avoid such poor design practices, the general controls that follow should be used where practical.

Consistent alignment should always be sought. Sharp curves should not be introduced at the ends of long tangents. Sudden changes from areas of flat curvature to areas of sharp curvature should be avoided. Where sharp curvature is introduced, it should be approached, where practical, by a series of successively sharper curves.

- **Broken-Back Curve.** A broken-back curve consists of two curves in the same direction joined by a short tangent (under 1,500 feet). Broken-back curves are undesirable and can typically be replaced by one simple curve. If used, a simple curve, a compound curve or spiral transitions should be used to provide some degree of continuous superelevation. Lengths need to be adequate to transition superelevation correctly. The “broken-back” arrangement of curves should be avoided except where very unusual topographical or right of way conditions make other alternatives impractical.
- **Small Deflection Angles.** For small deflection angles, curves should be sufficiently long enough to avoid the appearance of a kink. Curves should be at least 500 feet long for a central angle of 5 degrees, and the minimum length should be increased 100 feet for each 1 degree decrease in the central angle. Horizontal curves should not be used when the central angle is 59 minutes or less on non-freeways. The minimum length for horizontal curves on roadways should be fifteen times the design speed (15V). On higher-speed access-controlled facilities that use flat curves for aesthetic reasons, the desired minimum length for curves should be 30 times the design speed (30V), where length of curve (L) is in feet and design speed (V) is in mph.
- **Passing Tangents.** Passing tangents are used to provide passing opportunities on two-lane roads. One-half mile is considered an adequate length. Passing tangents should be provided as frequently as possible in keeping with the terrain.

An effort to introduce a passing tangent or to increase the length of a passing tangent is desirable if the project's budget can accommodate this addition. Nothing is gained by using large radii at the ends of a tangent if they reduce its length to less than that required for safe passing. It is better to use somewhat shorter radii and increase the intervening tangent to a more satisfactory length. At the other extreme, sharp curves at the ends of a passing tangent should be avoided as indicated above.



Example PBD Application

A designer is considering whether increasing the radius of a curve to meet standards is warranted. The existing condition curve does not meet standards but increasing curve radius to meet standards will be challenging considering the topographic and cost realities. Data, such as speed and volume, as well as crash patterns, can help determine the optimal radius for a given curve.

The designer contacts the Region Traffic Representative to access traffic engineering expertise in the area of Data Driven Safety Analysis (DDSA). The designer provides the traffic engineer the estimated cost associated with expanding the curve to meet standards. The traffic engineer leverages DDSA software with Colorado specific crash modification factors (CMF) to quantify the predicted safety performance of this specific curve location if it is left as is versus if the curve were modified to meet full standards. This key analysis then feeds into the benefit-cost analysis of the existing condition curve versus full standard curve scenarios. The designer then uses this analysis to inform the decision whether to modify the curve in conjunction with CSS considerations and other factors. Beyond this example involving curve radius, the designer should understand what other geometric elements can be similarly analyzed to make geometric design decisions.

6.3.5 Superelevation

6.3.5.1 General

One of the most important factors to consider in roadway safety is the centripetal force generated when a vehicle traverses a curve. Centripetal force increases as the velocity of the vehicle and/or the degree of curvature increases.

To overcome the effects of centripetal force, curves must be superelevated. It is impossible to balance centripetal force by superelevation alone because for any given curve radius, a certain superelevation rate is exactly correct for only one driving speed. At all other speeds, there will be a side thrust either outward or inward, relative to the curve center, which must be offset by side friction. Refer to Chapter 3, Section 3.3 of the 2018 AASHTO GDHS for further discussions on side friction.

Refer to Section 6.5 of this Guide for superelevation of detours.

6.3.5.2 Standards for Superelevation

The CDOT Standard Plans - M & S Standards on Superelevation (CDOT, 2019) give the required rate of superelevation for the various radius lengths at different design speeds for the maximum superelevation rate. Refer to CDOT Standard Plans - M & S Standards M-203-11, and M-203-12. The tables in the M&S Standards use Method 5 and are equivalent to 2018 AASHTO GDHS Tables 3-8, 3-9, and 3-10. Method 2 may be used for superelevation distribution for facilities in urban areas.

In general, the highest superelevation rate used on roadways in climates with snow and ice should be 8%. In practice, the maximum superelevation rate chosen on Colorado highways is typically either 6% or 8% after the designer considers the four factors discussed in Chapter 3, Section 3.3.3, of the 2018 AASHTO GDHS.

While a maximum superelevation rate of 8% is generally practical elsewhere, a maximum superelevation rate of 6% is typically chosen in urban areas. The selection of 6% as the maximum superelevation rate is also common on viaducts where freezing and thawing conditions are likely because bridge decks generally freeze more rapidly than other roadway sections. Where roadways are intermittently elevated on bridges and viaducts, the lower superelevation rate should be used throughout for design consistency.

The maximum superelevation rate may be less than shown on CDOT Standard Plans - M & S Standards (CDOT, 2019) when the designer determines that the lower rate is required because of traffic congestion or extensive marginal development that act to restrict top speeds.

For divided roadways where the median width is less than 60 feet, future inside widening of bridges or providing additional lanes requires the designer to properly plan the superelevation. Things to consider are:

- Superelevation pivot point.
- Vertical clearance.
- Superelevation transitions.

6.3.5.3 Superelevation Transition

Superelevation runoff is the term denoting the length of roadway needed to accomplish the change in cross slope from a section with the adverse crown removed (one side superelevates at normal crown slope, the other side at zero slope) to the fully superelevated section, or vice versa. When a spiral is used, its length is used to accommodate the superelevation runoff.

Tangent runout is the term denoting the length of roadway needed to accomplish the change in cross slope from a normal crown section to a section with the adverse crown removed (one side superelevates at normal crown slope, the other side at zero slope), or vice versa.

The length of the tangent runout is determined by the amount of adverse crown to be removed and the rate at which it is removed. This rate of removal should be the same as the rate used to affect the superelevation runoff.

The location, with respect to the curve, and the various lengths of the superelevation transitions are shown on the CDOT Standard Plans - M & S Standards on Superelevation (CDOT, 2019).

6.3.5.4 Design for All Rural Roadways, Urban Freeways and High-Speed Urban Streets

On most roadways where speed is relatively high and relatively uniform, horizontal curves are generally superelevated and successive curves are generally balanced to provide a smooth riding transition from one curve to the next.

Refer to Chapter 3, Section 3.2.3.2, of the 2018 AASHTO GDHS, for the recommended maximum superelevation rates for all rural roadways, urban freeways, and high-speed urban streets.

6.3.5.5 Design for Low-Speed Urban Streets

Although superelevation is advantageous for traffic safety on higher speed, continuous flow roadways, various factors often combine to make its use impractical in many built-up areas. Such factors include wider pavement areas; need to match the grade of adjacent property; surface drainage considerations; pedestrian mobility challenges; and frequency of cross streets, alleys and driveways. Therefore, horizontal curves on low-speed streets in urban areas are frequently designed without superelevation, counteracting the centrifugal force solely with side friction. On these curves, traffic entering a curve to the left has an adverse or negative superelevation due to the normal crown, but with flat curves and lower speeds the resultant friction required to counteract both the centrifugal force and the negative superelevation is small.

On successively sharper curves for the same design speed, the maximum degree of curvature or sharpest curve without superelevation is reached when the side friction factor developed to counteract centripetal force and adverse crown reaches the maximum allowable value based on safety and comfort considerations. For travel on sharper curves, superelevation is needed.

The maximum superelevation rate of -2% (normal crown) in Table 6-2 on the next page establishes the minimum radius for each speed below which superelevation is not provided on local streets in residential and commercial areas but could also be considered for reverse curves at continuous flow intersections and special circumstances to accommodate proper drainage of the roadway. A maximum superelevation rate of 4% or 6% is commonly used. The cross slope of the roadway will determine the minimum radii to be used in Table 6-2. An adverse cross slope requires a larger radius. The maximum curvature for a given design speed is defined for low-speed urban streets when both the maximum superelevation rate and the maximum allowable side friction factors are utilized.

Refer to Chapter 3, Section 3.3.6, of the 2018 AASHTO GDHS for more information.

6.3.6 Widths for Turning Roadways at Intersections

Refer to Chapter 8 of this Guide for design of intersections.



Table 6-2 Minimum Radii and Superelevation for Low-Speed Urban Streets

| e (%) | R (ft) for Design Speed (mph) | | | | | | |
|----------|-------------------------------|-----|-----|-----|-----|-----|------|
| | 15 | 20 | 25 | 30 | 35 | 40 | 45 |
| -2.0 | 50 | 107 | 198 | 333 | 510 | 762 | 1039 |
| -1.5 | 49 | 105 | 194 | 324 | 495 | 736 | 1000 |
| 0 | 47 | 99 | 181 | 300 | 454 | 667 | 900 |
| 1.5 | 45 | 94 | 170 | 279 | 419 | 610 | 818 |
| 2.0 | 44 | 92 | 167 | 273 | 408 | 593 | 794 |
| 2.2 | 44 | 91 | 165 | 270 | 404 | 586 | 785 |
| 2.4 | 44 | 91 | 164 | 268 | 400 | 580 | 776 |
| 2.6 | 43 | 90 | 163 | 265 | 396 | 573 | 767 |
| 2.8 | 43 | 89 | 161 | 263 | 393 | 567 | 758 |
| 3.0 | 43 | 89 | 160 | 261 | 389 | 561 | 750 |
| 3.2 | 43 | 88 | 159 | 259 | 385 | 556 | 742 |
| 3.4 | 42 | 88 | 158 | 256 | 382 | 550 | 734 |
| 3.6 | 42 | 87 | 157 | 254 | 378 | 544 | 726 |
| 3.8 | 42 | 87 | 155 | 252 | 375 | 539 | 718 |
| 4.0 | 42 | 86 | 154 | 250 | 371 | 533 | 711 |
| 4.2 | 41 | 85 | 153 | 248 | 368 | 528 | 703 |
| 4.4 | 41 | 85 | 152 | 246 | 365 | 523 | 696 |
| 4.6 | 41 | 84 | 151 | 244 | 361 | 518 | 689 |
| 4.8 | 41 | 84 | 150 | 242 | 358 | 513 | 682 |
| 5.0 | 41 | 83 | 149 | 240 | 355 | 508 | 675 |
| 5.2 | 40 | 83 | 148 | 238 | 352 | 503 | 668 |
| 5.4 | 40 | 82 | 147 | 236 | 349 | 498 | 662 |
| 5.6 | 40 | 82 | 146 | 234 | 346 | 494 | 655 |
| 5.8 | 40 | 81 | 145 | 233 | 343 | 489 | 649 |
| 6.0 | 39 | 81 | 144 | 231 | 340 | 485 | 643 |

Source: 2018 AASHTO GDHS.

Notes:
Computed using Superelevation Distribution Method 2.
Superelevation may be optional on low-speed urban streets.

6.3.7 Traveled Way Widening on Horizontal Curves

Curve widening is used primarily on pavements of substandard width or curvature to support the off tracking of larger design vehicles as described in the following sections. This can be especially important when bicycle lanes or shoulders are located adjacent to tighter horizontal curves and would otherwise require a larger vehicle to track into the path of a cyclist. On open roadway curves, the pavement should be widened as shown in Chapter 3, Section 3.3.10, of the 2018 AASHTO GDHS, which is based on a WB-62 design vehicle.

Refer to Chapter 3, Section 3.3.10, of the 2018 AASHTO GDHS for more information.

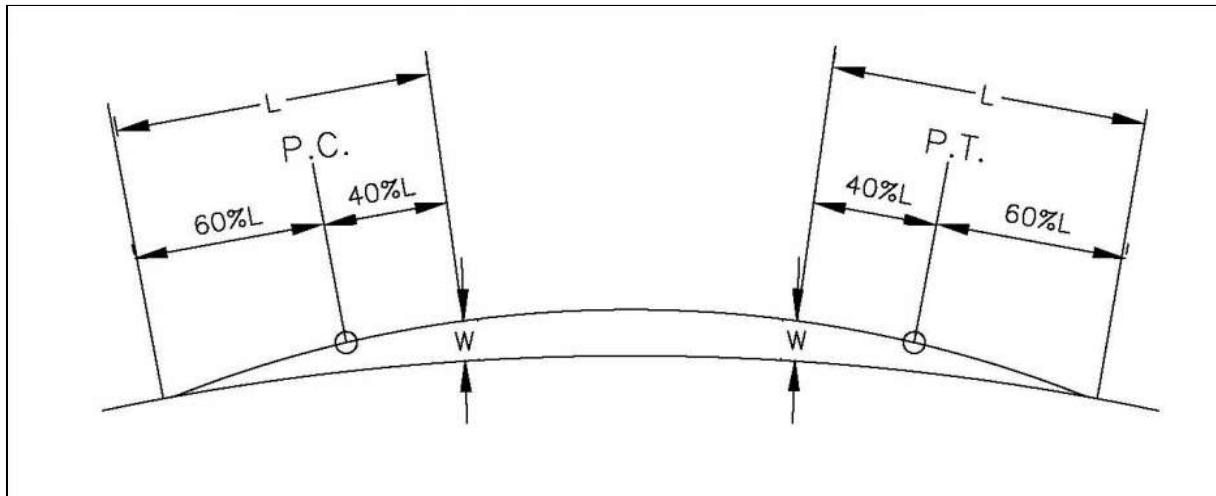


6.3.7.1 Attainment of Widening on Curves

Widening should be attained gradually on the approaches to a simple curve and spiral curve, as shown in Figure 6-10 and Figure 6-11 respectively, to ensure a reasonably smooth alignment of the edge of the pavement and to fit the paths of vehicles entering or leaving the curves.

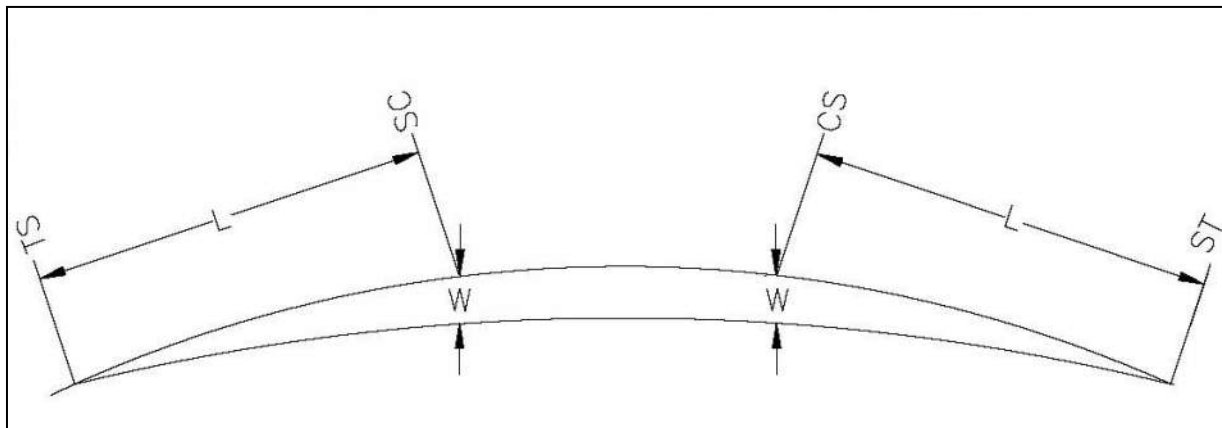
Widen (W) on the inside edge of the pavement and extend the transition over the same transition length (L) as the superelevation runoff.

Figure 6-10 Widening on a Simple Curve



Source: Section 6C.08 of the MUTCD.

Figure 6-11 Widening on a Spiral Curve



Source: Section 6C.08 of the MUTCD.

When $V \leq 40$ mph $L \geq \frac{WV^2}{60}$

When $V > 40$ mph $L > WV$



Where:

W = Width of widening (ft)

V = Design Speed (mph)

Example:

Given:

Pavement = 22 feet

Degree of curve = 9°

Radius = 636.62 feet

Icing conditions frequently exists, crowned highway

When V=30 mph

From Table 3-26b of the GDHS (1), Widening (W) = approximately 3.5 feet

From CDOT *Standard Plans - M&S Standards (3)*, Superelevation

Crowned Highways, L=100 feet

Since $V \leq 40$, $\frac{WV^2}{60} = \frac{3.5(30)^2}{60} = 52.5$ feet

And $L > \frac{WV^2}{60}$ OK

When V=50 mph and curve radius = 760 feet with other parameters same as above,

From Table 3-36b of the GDHS (1), Widening (W) = approximately 4.3 feet

From CDOT *Standard Plans - M&S Standards (3)*, Superelevation

Crowned Highways, L= 240 feet

Since $V > 40$ mph, $WV = 4.3(50) = 215$ feet

And $L > WV$ OK

6.3.8 Transition Design Controls

Refer to Chapter 3, Section 3.3.8, of the 2018 AASHTO GDHS for more information.

6.3.9 Pavement Transitions

6.3.9.1 General

A pavement transition is the area of variable pavement width encountered when changing from one roadway width, or section, to another.

6.3.9.2 Two Lanes to Multilane Divided

This type of transition should be made only where sight distance is not restricted, such as on a tangent section or on a flat curve. On a tangent section, the transition may be accomplished on either one or both lanes. A maximum of 1 degree reversing curves and a minimum total transition length of 1,000 feet should be used. This minimum length shall also apply where the transition is accomplished on a curve.

Design standards of the two lanes should be consistent with those of the multilane facility.

6.3.9.3 Other Transitions

Other, more simplified, transitions occur at speed-change lanes, truck climbing lanes, and widening for curves. All transitions shall be consistent with the design speed for the facility.

6.3.10 Off Tracking



As stated in Chapter 3, Section 3.3.9, of the 2018 AASHTO GDHS, “Off tracking is the characteristic, common to all vehicles, although it is much more pronounced with larger vehicles, where the rear wheels do not precisely follow the same path as the front wheels when the vehicle traverses a horizontal curve or makes a turn. When a vehicle traverses a curve without superelevation at low speed, the rear wheels track inside the front wheels. When a vehicle traverses a superelevated curve, the rear wheels may track inside the front wheels more or less than they do for a curve without superelevation. This is because of the slip angle of the tires with respect to the direction of travel, which is induced by the side friction developed between the pavement and rolling tires. The relative position of the wheel tracks depends on the speed and the amount of friction developed to sustain the lateral force not offset by superelevation or, when traveling slowly, by the friction developed to counteract the effect of superelevation not compensated by lateral force. At higher speeds, the rear wheels may even track outside the front wheels.”

Refer to Chapter 3, Section 3.3.9, of the 2018 AASHTO GDHS for more information.

6.3.11 General Controls for Horizontal Alignments

Refer to Chapter 3, Section 3.3.13 of the 2018 AASHTO GDHS for more information.

6.3.12 PBPD, MM, and CSS Considerations

Design options for determining a rock cut or widening the median on the I-70 Mountain Corridor included Context Sensitive Solutions (CSS) and PBPD considerations.

The walls in the median on the I-70 Mountain Express Lane have always been considered an interim solution. Clear Creek County stakeholders valued an open median area and did not want I-70 to feel like an urban corridor. However, because of the geology, widening to the rock side would have required blasting and cutting the rock back, resulting in a 20-foot widening instead of the 6-foot widening needed for the additional lane. The rock cut would be considered much more permanent than the walls in the median, which could be removed in the future.

6.4 Vertical Alignment

6.4.1 Terrain

The grade line is a reference line by which the elevation of the pavement and other features of the roadway are established. It is controlled mainly by topography and structure clearances, but the factors of horizontal alignment, safety, sight distance, design speed, construction costs, and the performance of heavy vehicles on a grade also must be considered.

In flat terrain, the elevation of the grade line is often controlled by drainage considerations.

In rolling terrain, an undulating grade line is often desirable, both from a standpoint of construction and maintenance economy. However, undulating grade lines involving substantial lengths of descending grades should be evaluated for their effect upon traffic operations and safety since they may result in undesirably high downgrade truck speeds.

In mountainous terrain, the grade line is usually closely dependent upon physical controls, although adverse grades should be avoided. On divided roadways, independent profiles with grade differential should be considered. Broken-back grade lines should always be avoided.

On long grades, it is preferable to flatten the grades near the top of the ascent particularly on low design speed roadways.

In all cases, the consideration of adequate sight distance requirements and other safety factors should take precedence over construction and maintenance costs.

6.4.2 Position with Respect to Cross Section

The grade line should generally coincide with the axis of rotation for superelevation.

- On undivided roadways, the grade line should coincide with the roadway centerline.
- On ramps and interstate-to-interstate connections, the grade line is generally positioned at the left edge of the traveled way. Either edge of traveled way or centerline may be used on multilane facilities.
- On divided roadways, the grade line should be positioned at the centerline of the median for paved medians 60 feet wide or less.

In selecting where the grade line is in relation to the axis of rotation for superelevation, the designer should consider the following:

- Future widening.
- Mountainous terrain.
- Right of way constraints.
- Topographic features.
- Earthwork.
- Matching existing typical sections (as-constructed plans).

Table 6-3 on the next page shows clearances to structures and obstructions for the various functional classifications within the right of way.

6.4.3 Standards for Grades

6.4.3.1 Minimum Grades

Flat and level grades on uncurbed pavements are acceptable when the pavement is adequately crowned to drain the surface laterally (refer to Chapter 4 of this Guide).

With curbed pavements, longitudinal grades should be sufficient to facilitate curb drainage. A minimum curb flowline grade for the usual case is 0.5%, but a grade of 0.3 % may be used where there is a high-quality pavement adequately crowned and supported on firm subgrade. With curbed sections on sag vertical curves, a grade of at least 0.30% should be retained at the curb and gutter line by increasing the crown slope or, if necessary, shortening the vertical curve length to keep the crown slope from exceeding the maximum value given in Chapter 4.

6.4.3.2 Maximum Grades

The desirable maximum grades for the various functional classifications are shown in Table 6-4. The maximum design grade should be used infrequently; in most cases, grades should be less than the maximum design grade.

The term "critical length of grade" is used to indicate the maximum length of a designated upgrade upon which a loaded truck can operate without an unreasonable reduction in speed. On grades longer than "critical," consideration of auxiliary climbing and decent lanes should be made.

Refer to Chapter 3, Section 3.4.2, of the 2018 AASHTO GDHS for more information.

6.4.4 Methods for Increasing Passing Opportunities on Two-Lane Roads

Passing lanes can reduce crash potential on two lane roads. The designers should look for opportunities to introduce passing lanes where feasible and based on the project purpose and need.

Refer to Chapter 3, Section 3.4.4, of the 2018 AASHTO GDHS for more information.



Table 6-3 Minimum Vertical Clearances Within Right of Way

| | HIGHWAY UNDERPASSES | | RAILWAY UNDERPASSES *** | | OVERHEAD WIRES | |
|-----------------------|---------------------|------------|-------------------------|-------------|----------------|-----------|
| | HORIZONTAL AL | VERTICAL* | HORIZONTAL | VERTICAL* ‡ | HORIZONTAL^ | VERTICAL* |
| Local Rural Roads | E | 15 feet | F | G | A | H |
| Local Urban Streets** | | | | | B | |
| Rural Collectors | | | | | C | |
| Urban Collectors** | | 16.5 feet# | | | D | |
| Rural Arterials | | | | | | |
| Urban Arterials** | | | | | | |
| Freeways | | | | | | |

- A 10 feet from edge of traveled way.
- B Use A when practical, but in any event, provide a minimum of 2 feet from curb face or from shoulder edge. See *AASHTO Interim Revisions to Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (AASHTO, 2020).
- C ≤40mph use A, ≥45mph use D.
- D Use a clear zone according to the *AASHTO Roadside Design Guide* (AASHTO, 2011) or provide guard rail.
- E Carry full approach roadway section through structure; minimum clearance from edge of through traffic lanes to walls, piers, or toes of slopes shall correspond to D, but, desirably, should not be less than 30 feet, unless guard rail is used. For tunnels and depressed roadways see the *AASHTO Standard Specifications for Highway Bridges* (AASHTO, 2012).
- F See *CDOT Bridge Design Manual* (CDOT, 2023).
- G Minimum requirements vary by railroad. BNSF requires 23.5 feet, UPRR requires 23.33 feet per UPRR-BNSF Guidelines for Grade Separation Projects-Rev 2016 (Union Pacific, 2016). PUC and federal minimum requirement is 23 feet per AREMA.
- H Communication lines and power lines in accordance with Table 1 of the *State Utility Accommodation Code* (CDOT, 2021):

| Type of Conductor, Cable & Voltage | Over Roadway Template (feet) | Outside Roadway Template (feet) |
|--|------------------------------|---------------------------------|
| Insulated communication conductors & cables; messengers; grounded or effectively insulated guys; effectively grounded neutral conductors; 230C1 supply cables. | 24 | 20 |
| Noninsulated communication conductors; supply cables 0-750 Volts (multiplex wire) | 24 | 20.5 |
| Open Supply Conductors 0-750 Volts | 24 | 21 |
| Open Supply Conductors >750 Volts to 22 kVolts | 25 | 23 |
| Voltages exceeding 22 kVolts to 50 kVolts | 25♦ | 23♦ |
| Voltages exceeding 50 kVolts | 25.5♦♦ | 23♦♦ |

- ♦ plus 0.4 inch per 1,000 Volts in excess of 22 kVolts
- ♦♦ plus [0.4 inch per 1,000 Volts in excess of 22 kVolts] X [1.0 + (.03 per 1,000 feet above 3,300 feet above sea level)] or alternate method for voltages exceeding 98 kVolts

Voltages are phase to ground for effectively grounded circuits and those other circuits where all ground faults are cleared by promptly de-energizing the faulted section, both initially and following subsequent breaker operations

Source CDOT Utility Accommodation Code (2021) (2 C.C.R. 601-18)



Table 6-4 Relation of Maximum Grades to Design Speed

| Type of Terrain | Maximum Grade (%) for Design Speed (mph) | | | | | | | | | | |
|--|--|----------------|----------------|----------------|----------------|----------------|----------------|----|----|----|----|
| | 20 | 30 | 40 | 45 | 50 | 55 | 60 | 65 | 70 | 75 | 80 |
| RURAL AND URBAN FREEWAYS ^a [2018 AASHTO GDHS Table 8-1] | | | | | | | | | | | |
| LEVEL | | | | | 4 ^c | 4 ^c | 3 ^c | 3 | 3 | 3 | 3 |
| ROLLING | | | | | 5 ^c | 5 ^c | 4 ^c | 4 | 4 | 4 | 4 |
| MOUNTAINOUS | | | | | 6 | 6 | 6 | 5 | 5 | -- | -- |
| RURAL ARTERIALS [2018 AASHTO GDHS Table 7-2] | | | | | | | | | | | |
| LEVEL | | | 5 ^c | 5 ^c | 4 ^c | 4 ^c | 3 | 3 | 3 | 3 | 3 |
| ROLLING | | | 6 ^c | 6 ^c | 5 | 5 | 4 | 4 | 4 | 4 | 4 |
| MOUNTAINOUS | | | 8 | 7 | 7 | 6 | 6 | 5 | 5 | 5 | 5 |
| URBAN ARTERIALS [2018 AASHTO GDHS Table 7-4] | | | | | | | | | | | |
| LEVEL | | 8 | 7 | 6 | 6 | 5 | 5 | | | | |
| ROLLING | | 9 | 8 | 7 | 7 | 6 | 6 | | | | |
| MOUNTAINOUS | | 11 | 10 | 9 | 9 | 8 | 8 | | | | |
| RURAL COLLECTORS ^b [2018 AASHTO GDHS Table 6-2] | | | | | | | | | | | |
| LEVEL | 7 ^c | 7 ^c | 7 | 7 | 6 | 6 | 5 | | | | |
| ROLLING | 10 ^c | 9 | 8 | 8 | 7 | 7 | 6 | | | | |
| MOUNTAINOUS | 12 | 10 | 10 | 10 | 9 | 9 | 8 | | | | |
| URBAN COLLECTORS ^b [2018 AASHTO GDHS Table 6-8] | | | | | | | | | | | |
| LEVEL | 9 ^c | 9 | 9 | 8 | 7 | 7 | 6 | | | | |
| ROLLING | 12 ^c | 11 | 10 | 9 | 8 | 8 | 7 | | | | |
| MOUNTAINOUS | 14 ^c | 12 | 12 | 11 | 10 | 10 | 9 | | | | |
| LOCAL RURAL ROADS [2018 AASHTO GDHS Table 5-2] | | | | | | | | | | | |
| LEVEL | 8 ^d | 7 | 7 | 7 | 6 | 6 | 5 | | | | |
| ROLLING | 11 | 10 | 10 | 9 | 8 | 7 | 6 | | | | |
| MOUNTAINOUS | 16 | 14 | 13 | 12 | 10 | 10 | -- | | | | |

- a Grades 1% steeper than the value shown may be used in urban areas.
- b Maximum grades shown for rural and urban collector conditions of short lengths (less than 500 feet) and on one-way downgrades may be 2% steeper.
- c Design speed shown not recommended (less than minimum).
- d Use only on urban streets.

6.4.5 Passing Lanes

Passing lanes on two-lane highways improve traffic operations on roadway sections of lower capacity and on lengthy sections with no passing opportunities (greater than 6 miles).

The logical location for a passing lane is where passing sight distance is restricted, but adequate sight distance should be provided at both the add and drop lane tapers. A minimum sight distance of 1,000 feet on the approach to each taper is recommended. The selection of the passing lane location should consider the location of intersections and high-volume driveways, as well as physical constraints such as bridges and culverts, that could restrict provision of a continuous shoulder.

Use the following design procedure to identify the need for passing sections on two-lane roadways:

1. Design horizontal and vertical alignment to provide as much of the roadway as practical with passing sight distance. Refer to the Passing Sight Distance column in Table 6-1.



2. Where the design volume approaches capacity, recognize the effect of lack of passing opportunities in reducing the level of service.
3. Determine the need for climbing lanes.
4. Where the extent and frequency of passing opportunities made available by application of Criteria 1 and 3 are still too few, consider the construction of passing lane sections.

Passing lane sections should be sufficiently long to permit several vehicles in a line behind a slow-moving vehicle to pass before returning to the normal cross section of two-lane roadway. The minimum length, excluding tapers, should be 1,000 feet. A lane added to improve overall traffic operations should be long enough, over 0.5 mile, to provide a substantial reduction in traffic platooning, as shown in Table 3-5. Consideration should also be given to implementing a pavement marking taper in accordance with the MUTCD that redirects all traffic into the outside travel lane as a way to encourage slower moving traffic to move to the right and allow drivers to complete their passing maneuver on the left.

Table 6-5 Optimal Passing Lane Lengths for Traffic Operations Efficiency

| One-Way Flow Rate (vehicles/hour) | Passing Lane Length (miles) |
|--------------------------------------|--------------------------------|
| 100-200 | 0.50 |
| 201-400 | 0.50-0.75 |
| 401-700 | 0.75-1.00 |
| 701-1200 | 1.00-2.00 |

Source: Table 3-31 of the 2018 AASHTO GDHS.

The transition tapers at each end of the added lane section should be designed to encourage safe and efficient operation. The lane drop taper should be computed from the MUTCD formulas below.

For $S \geq 45$ mph, $L = WS$

For $S < 45$ mph, $L = \frac{WS^2}{60}$

Where:

L = Length of taper (ft)

W = Width of lane (ft)

S = Posted Speed (mph)

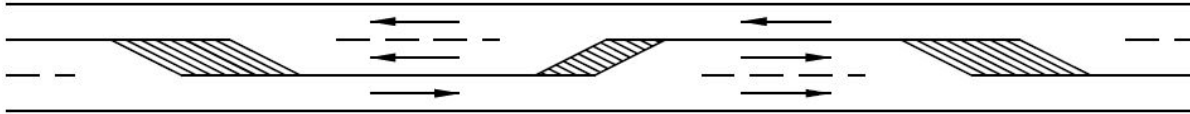
The recommended length for the lane addition taper is half to two-thirds of the lane drop length. The transitions should be located where the change in width is in full view of the driver.

6.4.6 2+1 Roadways

The 2+1 roadway concept has been found to improve operational efficiency and reduce crashes along some two-lane highways. The 2+1 concept provides a continuous three-lane cross section with alternating passing lanes (Figure 6-12). This configuration may be suitable for corridors with

traffic volumes higher than can be served with isolated passing lanes, yet not high enough to require a consistent four-lane cross section.

Figure 6-12 Schematic for 2+1 Roadways



Source: Figure 3-33 of the 2018 AASHTO GDHS.

A 2+1 roadway generally operates two levels of service higher than a conventional two-lane road with the same traffic volume. 2+1 roadways should not generally be considered where the volume exceeds 1,200 vehicles per hour in one direction. 2+1 roadways should be used on level or rolling terrain where climbing lanes should be used on mountainous terrain or steep grades as an alternative. Intersection locations should be considered when determining passing locations to minimize turning movements within passing lanes or to provide dedicated left-turn lanes at intersections.

6.4.7 Turnouts

It is not always economically feasible to provide passing lanes or continuous wide shoulders on roadways through deep rock cuts or where other conditions limit the cross section width. In such cases, consideration should be given to use of intermittent sections of shoulder or turnouts along the roadway. Such turnouts provide an area for emergency stops and allow slower-moving vehicles to pull out of the through lane so following vehicles can pass.

Turnouts should be located so that approaching drivers have a clear view of the entire turnout to determine whether the turnout is available for use. The sight distance for vehicles re-entering the road should also be reconsidered. Refer to Table 3-32 of the 2018 AASHTO GDHS for recommended lengths of turnouts, including taper.



Bicycle turnouts can be utilized where there is a lack of shoulder space. Refer to Chapter 13 of this Guide for design guidance.

6.4.8 Emergency Escape Ramps

Emergency escape ramps (runaway truck ramps) are widely used in Colorado because of the mountainous terrain and steep grades that are present throughout a large portion of the state. There are many long and descending grades that need to be accounted for with larger vehicles. For design elements, design guidelines, and more information refer to Chapter 3, Section 3.4.5, of the 2018 AASHTO GHDS.

6.4.9 Vertical Curves

Properly designed vertical curves should provide adequate sight distance, safety, comfortable driving, good drainage, and pleasing appearance.

Vertical curves are parabolic. Figure 6-13 gives the necessary mathematical relations for computing a vertical curve, at either crests or sags.

Minimum lengths of sag vertical curves are typically controlled by headlight sight distance that should be approximately the same as stopping sight distance. In areas with adequate overhead lighting where headlight sight distance is not a limitation, the driver comfort factor can be used to determine the minimum length of sag vertical curve. Sag vertical curve lengths satisfying the comfort factor are approximately 50% of that needed to satisfy headlight sight distance criterion.

Refer to Chapter 3, Section 3.4.6 of the 2018 AASHTO GDHS for more information.

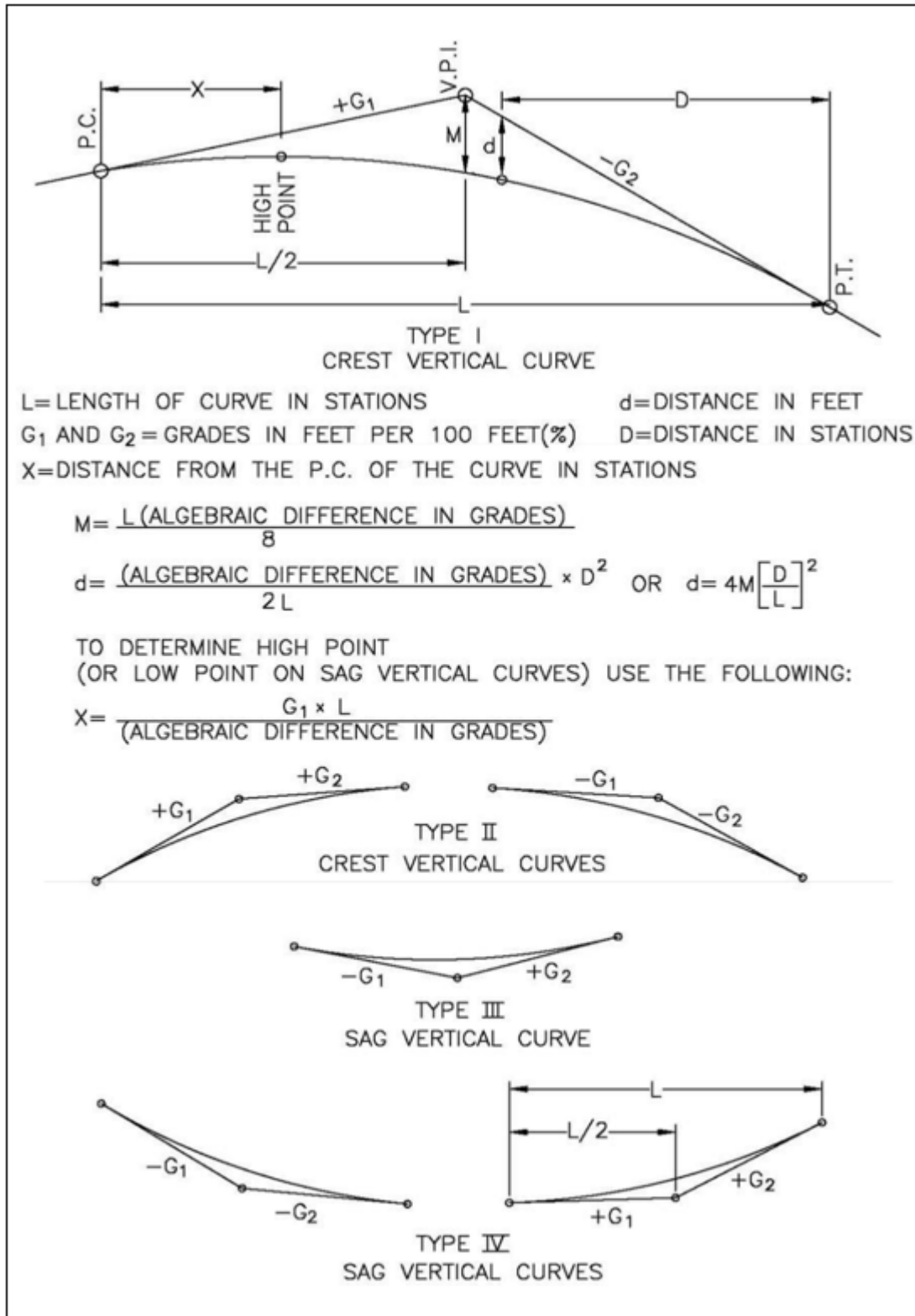
Vertical curves are not required where algebraic grade difference is less than 0.20%. In rural applications, the minimum length of vertical curves on main roadways, both crest and sag, should be 300 feet. For other applications, the minimum length should be about three times the design speed.

Vertical curves that have a level point and flat sections near their crest or sag should be evaluated for drainage where curbed pavements are used.

Values of $K = 167$ or greater should be checked for drainage. ($K = \frac{L}{A}$); where L is the length of curve in feet, and A is the algebraic difference in grade. K value is a coefficient by which the algebraic difference in grades (A) may be multiplied to determine the length in feet (L) of the vertical curve that will provide minimum sight distance.

Also, vertical curves that are long and flat may develop poor drainage at the level section. This difficulty may be overcome by adjusting the flow line of the ditch section.

Figure 6-13 Vertical Curves



6.4.10 Climbing Lanes

On long, steep grades, a climbing lane for the slow-moving vehicles may be required. Criteria for establishing the need for such lanes are usually based on traffic volume, capacities, percent of trucks, grades, speeds, and level of service. Because of many variables, no set of conditions can be properly described as typical. A detailed analysis should be made wherever climbing lanes are being considered. A discussion of the analytical approach to be followed is presented in Chapter 3-4 of the 2018 AASHTO GDHS.

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

- Upgrade traffic flow in excess of 200 vehicles per hour.
- Upgrade truck flow rate in excess of 20 vehicles per hour.
- One of the following conditions exist:
 - A 10 mph or greater speed reduction is expected for a typical heavy truck or recreational vehicle.
 - Level of service (LOS) E or F exists on the grade.
 - A reduction of two or more LOS is experienced when moving from the approach segment to the grade.

The width of the climbing lane should be the same width as the regular traffic lanes. The beginning of the climbing lane should be preceded by a tapered section with a taper ratio of 25:1, and at least 150 feet long.

Desirably, the shoulder on the outer edge of a climbing lane should be as wide as the shoulder on the normal two-lane section. Conditions, however, may dictate otherwise, particularly when the climbing lane is added to an existing roadway. A usable shoulder of 4 feet wide or greater is acceptable.



Where there is a known bicycle route on a long ascent, bicycle climbing lanes could be considered. In mountainous areas, a 4- to 6-foot bicycle climbing lane can provide operational and safety benefit. When combined with a 4-foot shoulder, the bicycle lane + shoulder can be used in an emergency for pull-off.

The ideal design would be to extend the climbing lane to a point beyond the crest, where a typical truck could attain a speed that is within 10 mph of the speed of the other vehicles with the desirable speed being at least 40 mph, approximately at a LOS D. Even this may not be practical in many instances because of the unduly long distance required for trucks to accelerate to the desired speed. For such a condition, the climbing lane should end at a point where the truck can return to the normal lane without creating undue hazard. This would be feasible where the sight distance becomes sufficient to permit passing with safety when there is no oncoming traffic or

preferably at least 200 feet beyond this point. In addition, a corresponding length of taper should be provided to permit the truck to return to the normal lane.

For example, on a roadway where the safe passing sight distance becomes available 100 feet beyond this point, the truck lane should extend 100 feet:

- Plus 200 feet or 300 feet beyond the crest,
- Plus an additional length for taper, preferably at a ratio of 50:1 but with a taper length of at least 200 feet.

Figures 3-24 and 3-25 of the 2018 AASHTO GDHS show the relationship between rate and length of grade for several reductions in speed. The 10 mph speed reduction curve is used as the design guide.



Climbing lanes can also be a bicycle facility, particularly on two-lane Roads in the C1 context classification. Refer to Chapter 13 of this Guide for design details.

The method for determining passing lane location is described in Section 3.4.3 (page 3-125) of the 2018 AASHTO GDHS.

6.5 Combination of Horizontal And Vertical Alignment

Refer to Chapter 3, Section 3.5, of the 2018 AASHTO GDHS for more information.

To avoid the possibility of introducing serious hazards, coordination is required between horizontal and vertical alignment. Particular care must be exercised to maintain proper sight distance. Where grade line and horizontal alignment will permit, it is desirable to superimpose vertical curves on horizontal curves. This reduces the number of sight distance restrictions and makes changes in the profile less apparent, particularly in rolling country. Care should be taken, however, not to introduce a sharp horizontal curve near a pronounced summit or grade sag. This is particularly hazardous at night.

Horizontal curvature and profile grade should be made as flat as possible at roadway intersections.

On divided roadways, variation in the width of median and the use of separate profiles and horizontal alignment should be considered to derive the design and operational advantages of one-way roadways.

6.6 Guidelines For Designing Detours

For the purpose of applying these guidelines, a detour is any temporary routing of traffic off its usual course, including the use of existing alternate routes or use of modified lane widths on the main roadway.

The following criteria guidelines are recommended when designing a detour. Items are those that must be addressed when requesting detour approval. This is not a design policy, and circumstances may often justify departure from these guidelines. For further reference, refer to CDOT Form 518.

6.6.1 Detour Design Speed

The design speed of a detour should be as close to the mainline operating speed as possible. Every effort should be made to keep the speed differential within 10 mph so as not to affect the safety and operation of the facility, although in some cases a maximum of 15 mph or more may be necessary. As truck traffic increases so should the emphasis on providing the lowest possible speed differential. An exception may be posted city streets. Refer to Chapter 3 of this Guide.

6.6.2 MM, PBPD, and CSS Considerations

6.6.2.1 Pedestrian Accommodations

To the extent possible, construction impacts on sidewalks and pedestrian ramps should be minimized. Where sidewalk narrowing or closure is unavoidable, pedestrians should be provided with a convenient and accessible path that replicates as nearly as practical the existing facility. If the temporary traffic control affects an accessible and detectable pedestrian facility, accessibility and detectability shall be maintained along the alternate pedestrian route (refer to MUTCD 6D). This includes providing a smooth continuous hard surface, detectable channelization or barriers, and ramps at grade changes.

When sidewalks are impacted by construction, the best solution is to accommodate pedestrians with a sidewalk diversion as opposed to a detour. A sidewalk diversion is a safe, protected, accessible route that runs parallel to the existing sidewalk, typically utilizing the adjacent parking lane, travel lane or roadway shoulder with a detectable and compliant vertical barrier. Sidewalk diversions provide a predictable, accessible route that minimizes the additional distance pedestrians must walk and eliminates pedestrians walking in active vehicle travel lanes instead of using a detour. This is especially true on high traffic volume arterials that may not have sufficient pedestrian crossings to provide a reasonably short detour route and may increase instances of pedestrians crossing mid-block at uncontrolled locations.

If unavoidable, sidewalk detours should meet the same accessibility and detectability requirements as a sidewalk diversion. Special care should be paid to advance notification of sidewalk closures (with detectable edges on all signage and barriers). Audible information devices may be used to verbally describe detour routes to pedestrians with visual disabilities.

6.6.2.2 Transit Accommodations

To the extent possible, construction impacts to transit stops should be minimized. Where possible, an ADA accessible temporary facility can be implemented in a parking lane to provide uninterrupted transit service. If no other alternative is feasible, the work zone designer should work with the transit operator to temporarily relocate the transit stop outside of the work zone

within a reasonable distance of the existing stop, and accessible routes to the stop should be provided if not already in place.

6.6.2.3 Bicycle Accommodations

To the extent possible, construction impacts on linear bicycle facilities such as bicycle lanes and shared use paths should be minimized. The context of the bicycle facility is critical to designing safe and effective temporary traffic control plans, with urban and rural contexts requiring differing approaches based on traffic volumes and speeds.

For all contexts, if impacts to linear bicycle facilities are necessary for project construction, then a viable alternative should be provided immediately adjacent to the existing facility where possible. Temporary bicycle facilities should maintain a similar quality as the existing facility (width, protection, surface) which may require removal of parking lanes, underutilized traffic lanes or shoulders. Narrowing of parking lanes, traffic lanes or shoulders should also be considered. Maintaining separate temporary bicycle facilities is critical where high traffic volumes and speeds would make a shared roadway treatment unsafe. If an on-roadway separate facility is infeasible on high speed/volume roadways, an off-roadway temporary path may be provided if space is available. In situations where a shared roadway or signed bicycle route requires flaggers for an alternating one-way operation, processing bicycle and vehicle queues separately will maintain separation of users. Detour routes or vehicle shuttles should only be employed when necessary for safety reasons (especially in rural contexts where distances may be greater) and should consider detour length, grade, shoulder width, and surface condition (grates, surface type, etc.)

6.6.3 Detour Clear Zone

Criteria corresponding to the speed, geometry, and traffic of the existing roadway should be used to design the detour. Detour culverts should be included in the clear zone analysis. For more information refer to Chapter 9 of the 2011 AASHTO RSDG.

Portable barriers may often be the most cost-effective method of resolving detour clear zone problems.

6.6.4 Detour Typical Sections

Lane Width. It is desirable to maintain the width of the main roadway, but if this is not practical, the following guidelines apply:

- A minimum lane width of 10 feet may be used if all of the following conditions are satisfied:
 - The truck annual average daily traffic (AADT) is less than 50.
 - The design speed is ≤ 45 mph.
 - No curves are greater than 7 degrees.
- If one or more of the above conditions fails, 11-foot or wider lanes should be used.

- If any of the following conditions apply, 12-foot lanes should be used:
 - Design speed of 55 mph or more.
 - The truck AADT is greater than 300.
 - The road is classified as an arterial or on an arterial truck network system.
- If main roadway lanes are 11 feet, the detour may retain 11-foot width.

Shoulder Width. Desirable shoulder width is 4 feet. Two feet minimum is required.

6.6.5 Detour Barrier

A detour barrier is required when there are any hazards within the clear zone, including drop-offs or steep slopes. It may also be required for the protection of workers. Shoulder drop offs 3 inches or greater shall be mitigated within 24 hours.

When a barrier is used, it shall be installed at least 2 feet offset from edge of pavement with an appropriate distance from back of barrier to obstruction (refer to CDOT Standard Plans - M & S Standards [CDOT, 2019]). If shoulders are not provided, the barrier shall be installed 2 feet from the edge of the traveled lane. Where the situation allows, an offset of 4 feet from the travel lane should be provided.

6.6.6 Detour Surfacing

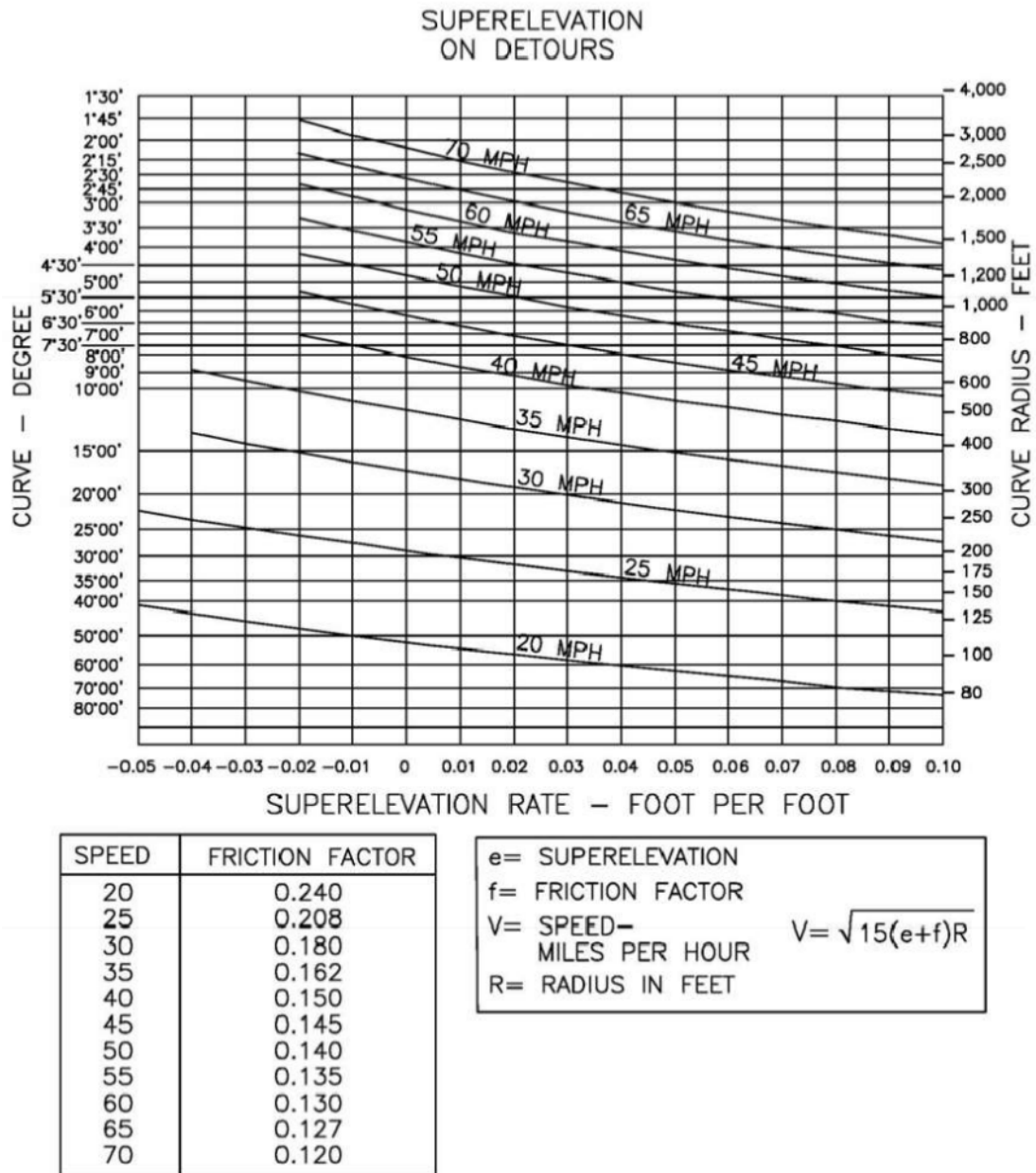
An asphalt surface is usually functionally superior to gravel, although gravel may have economic and other advantages. Asphalt should be used if detour speed is over 40 mph or the detour will be used for three weeks or more. Consult Region Materials Engineer for detour pavement design. Refer to the *CDOT Project Development Manual* (CDOT, [2013] 2022) for additional information.

6.6.7 Detour Superelevation

Figure 6-14 gives the rate of superelevation to be used on detours during construction of culverts, bridge replacement or widening, or repairs when proper construction signing is in place.



Figure 6-14 Superelevation on Detours



The formula shown in Figure 6-14 is the same as used in CDOT Standard Plans - M & S Standards (CDOT, 2019) shows superelevation and curvature for various design speeds, except that the "e" value shown in CDOT Standard Plans - M & S Standards (CDOT, 2019) is based on maximum driving comfort and safety, combined with a widely variable friction factor due to the potential for adverse pavement surface and weather conditions during construction.

Using Figure 6-14, the designer can choose a combination of friction factor, superelevation, and curvature to meet required design speed without the necessity of building up an excessive amount of superelevation and runoff which must be removed after a short time.

Values on Figure 6-14 have been checked by the "Ball Bank Indicator" to determine the point of discomfort for safe speeds on curves.

6.6.8 Detour on Local Roads

When local roads are used in detour routing, the stabilization needs must be reviewed. If necessary, additional overlay should be placed to protect the structural integrity of the street.

All the above-listed design elements, including the information on "Detour Design Data," shall be specified in the plans.

The following should be considered:

- Intergovernmental Agreement.
- Weight limits.
- Noise.
- Vibration.
- Traffic.
- Schools.
- Land-use.
- Pedestrian and bicycle use.

6.6.9 Environmental Considerations of Detours

When designing detours, it is important to consider and mitigate any possible environmental impacts. These can include wetlands, archaeology or paleontology resources, hazardous waste, water quality, or Section 4(f) involvement. These impacts may be avoided by the proper placement of the detour. For assistance in evaluating possible impacts, contact the Region Planning/ Environmental Section.

6.6.10 Detour Transverse Underdrains

Transverse underdrains are those constructed perpendicular to roadway. Refer to the *CDOT Drainage Design Manual* (CDOT, 2019).

6.7 Other Elements Affecting Geometric Design

6.7.1 Drainage and Erosion Control

Consider the following:

- Collect water prior to transitioning superelevation to prevent sheet flow.

- Design and locate inlets to limit the spread of water on the traveled way to tolerable widths.
- Install extra inlets near low points of sag vertical curves to take any overflow from blocked inlets.
- Locate inlets just upgrade of pedestrian crossings.
- Address environmental issues such as erosion and sediments.
- Dikes in medians and on the edge of the road should comply with clear zone requirements.
- Coordinate NPDES issues with Region Planning and Environmental Manager (refer to the *CDOT Project Development Manual* (CDOT, [2013] 2022).

Also refer to the *CDOT Drainage Design Manual* (CDOT, 2019).

6.7.2 Rest Areas

For design of rest areas, coordinate with CDOT's Property Management group and the FHWA Operations Engineer. Refer to the *AASHTO Guide for Development of Rest Areas on Major Arterials and Freeways* (AASHTO, 2001).

6.7.3 Lighting

Consider the following:

- Refer to the *Lighting Design Guidelines for the Colorado Department of Transportation* (CDOT, 2019).
- Coordinate with the Region Utility Engineer and the local utility company.
- Minimize light pollution in conformance with Colorado Revised Statutes (CRS) 24-82-902.
- Safety enhancement.

6.7.4 Utilities

Consider the following:

- Coordinate with the Region Utilities Engineer early and throughout the design process.
- Pothole to locate utilities as practical.
- Plot existing utilities in plan, profile, and cross sections to identify potential conflicts with design elements.
- Utility Notification Center of Colorado will not locate CDOT-owned utilities; contact the Region Traffic Signal Supervisor.
- Utility relocation requirements should be compatible with construction phasing.
- An Intergovernmental Agreement (IGA) may be necessary.

The clear zone dimensions to be maintained for a specific functional classification are discussed in the Section 4.6.1 of the 2018 AASHTO GDHS.

Utilities that are to cross or otherwise occupy the right of way of rural or urban freeways should conform to the AASHTO *A Policy on the Accommodation of Utilities Within Freeway Right-of-Way* (AASHTO, 2005a). Those on non-controlled access roadways and streets should conform to the AASHTO *A Guide for Accommodating Utilities Within Highway Right-of-Way* (AASHTO, 2005b).

6.7.5 Traffic Control Devices

The development of traffic control plans is an essential part of the overall project design and may affect the design of the facility itself. Refer to Chapter 15 of this Guide and the MUTCD.

6.7.6 Clear Zone

Refer to the AASHTO *Roadside Design Guide* (AASHTO, 2011) for more information.

6.7.7 PBPD, MM, and CSS Considerations

Picture a scenic road in a rural context surrounded by trees. This route is enjoyed by visitors and the local community alike in its current form. This scenic route currently has two 16-foot lanes with no accommodation for bicyclists although it is a popular cycling route. The scope of improvements of this corridor project places it in the 4R category. The design team is curious whether PBPD, MM and CSS considerations can be leveraged to arrive at the optimal design solution.

From a CSS perspective, it is clear the local stakeholders want the trees to stay. Unfortunately, many of the trees are within the clear zone.

While some stakeholders are not in favor of bike lanes, there is a concerning pattern of vehicle-bicycle crashes developing.

The design team decides to coordinate with their Region Traffic Representative regarding the need for Data Driven Safety Analysis (DDSA), a subtopic of PBPD. The goal of the DDSA will be to understand the safety performance impact of choosing a reduced clear zone where the vast majority of the trees can stay or where minimal guardrail will be required. The design team is also seeking to understand the safety performance impact of providing 11-foot driving lanes with 5-foot bike lanes. They wonder if the DDSA will reveal the predicted number of crashes for the reduced clear zone scenario will be elevated compared to the full clear zone scenario. They are also curious whether the predicted number of crashes for the 11-foot driving lane scenario will be elevated compared to the 16-foot lane scenario.

Additional design considerations (e.g., adjusting posted speed limit) or safety countermeasures may need to be applied to mitigate the potential increase in expected crashes.