PDHonline Course C793 (5 PDH)

Roadway Horizontal Alignments

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Gregory J. Taylor, P.E.

INTRODUCTION

The **roadway horizontal alignment** is a series of horizontal tangents (straight roadway sections), circular curves, and spiral transitions. It shows the proposed roadway location in relation to the existing terrain and adjacent land conditions. Together with the vertical alignment (grades and vertical curves) and roadway cross-sections (lanes, shoulders, curbs, medians, roadside slopes, ditches, sidewalks), the horizontal alignment (tangents and curves) helps to provide a three-dimensional roadway layout.

This course focuses on the geometric design of **horizontal alignments** for modern roads and highways. Its contents are intended to serve as guidance and not as an absolute standard or rule.

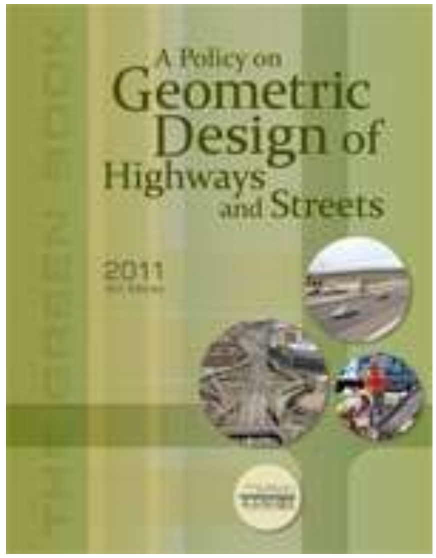
Upon course completion, you should be familiar with the general design of horizontal roadway alignments. The course objective is to give engineers and designers an in-depth look at the principles to be considered when designing horizontal alignments.

Subjects covered include:

- Sight Distance
  - *Stopping*
  - *Decision*
  - *Passing*
  - *Intersection*
- Design Considerations
  - *Cross slopes*
  - *Superelevation*
  - *Radii*
  - *Grades*
- Horizontal Curves
  - *Compound*
  - *Spiral*
- Coordination of Horizontal & Vertical Curves

A Policy on Geometric Design of Highways and Streets (also known as the "Green Book") published by the American Association of State Highway and Transportation

Officials (AASHTO) is considered to be the primary guidance for U.S. roadway design. For this course, Chapter 3 (Section 3.3 Horizontal Alignment) will be used exclusively for fundamental roadway geometric design principles.



BACKGROUND

Roadway geometric design consists of the following fundamental three-dimensional features:

Vertical alignment - grades and vertical curves

Horizontal alignment - tangents and curves

Cross section - lanes and shoulders, curbs, medians, roadside slopes and ditches, sidewalks

Combined, these elements contribute to the roadway's operational quality and safety by providing a smooth-flowing, crash-free facility.

Engineers must understand how all of the roadway elements contribute to overall safety and operation. Applying design standards and criteria to 'solve' a problem is not enough.

The fundamental objective of good geometric design will remain as it has always been **– to produce a roadway that is safe, efficient, reasonably economic and sensitive to conflicting concerns.**

HORIZONTAL ALIGNMENT

The **horizontal alignment** is a series of horizontal tangents (straight roadway sections), circular curves, and spiral transitions used for the roadway's geometry. This design shows the proposed roadway location in relation to the existing terrain and adjacent land conditions. The main objective of geometric roadway design is to integrate these elements to produce a compatible speed with the road's function and location. Safety, operational quality, and project costs can be significantly influenced by coordinating the horizontal and vertical alignments.

DESIGN SPEED

AASHTO defines design speed as "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern". It is an **overall design control** for horizontal alignments in roadway design that may equal or exceed the legal statutory speed limit. The level of service is directly related to the speed of operation - it should meet driver expectations and be consistent with the facility's functional classification and location.

Design speed selection is a critical decision that should be done at the beginning of the planning and design process. This speed should balance safety, mobility, and efficiency with potential environmental quality, economics, aesthetics, social and political impacts. Roadway design features (curve radii, superelevation, sight distance, etc.) are impacted by the design speed, as well as other characteristics not directly related to speed. Therefore, any changes to design speed may affect many roadway design elements.

Design speeds for **rural roads** should be as high as practicable to supply an optimal degree of safety and operational efficiency. Data has shown that drivers operate quite comfortably at speeds that are higher than typical design speeds.

Lower design speeds may be appropriate for certain **urban roadways** (residential streets, school zones, etc.). Traffic calming techniques have proven to be a viable option for residential traffic operations. Designers should evaluate high speed compatibility with safety (pedestrians, driveways, parking, etc.) for urban arterials.

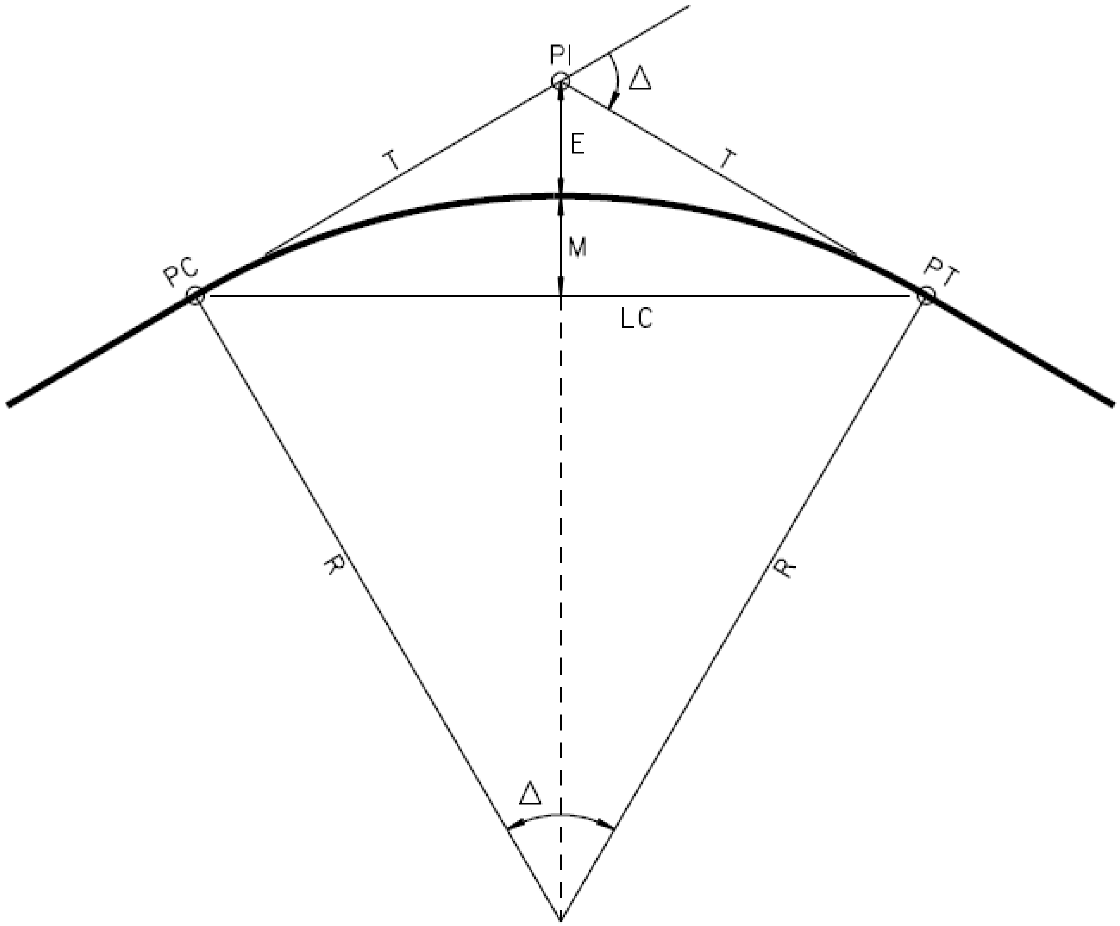
HORIZONTAL CURVES

Roadway horizontal curve design is based on the laws of physics and driver reaction to lateral acceleration. Any geometric alignment needs to address curve location; curve sharpness; tangent lengths; and how they relate to the vertical profile. All of these components should be balanced to operate at appropriate speeds under normal conditions.

Elements of Curve Design

- Curve radius
- Superelevation
- Side friction
- Assumed vehicle speed

Horizontal curves depend on specific values for a minimum radius (based on speed limit), curve length, and sight obstructions (sight distance). An increased superelevation (bank) may be required to assure safety for high speed locations with small curve radii. Designers must confirm sufficient sight distance around corners or curves in order to avoid crashes.



TERMS

R = Radius
PC = Point of Curvature (point at which the curve begins)
PT = Point of Tangent (point at which the curve ends)
PI = Point of Intersection (point at which the two tangents intersect)
T = Tangent Length (distance from PC to PI or PI to PT)
LC = Long Chord Length (straight line between PC and PT)
L = Curve Length (distance from PC to PT measured along the curve)
M = Middle Ordinate (distance from midpoint of LC to midpoint of the curve)
E = External Distance (distance from vertex to curve)
Δ= Deflection Angle (change in direction of two tangents)

The upper limits for superelevation on horizontal curves address constructability, land usage, slow-moving vehicles, and climate. For regular snow or ice locations, the superelevation should not exceed rates where slow-moving vehicles would slide toward the center of the curve. Hydroplaning can occur at high speed locations with poor drainage that allow a build-up of water.

SIDE FRICTION FACTOR

A vehicle's need for side friction (*side friction demand*) is represented by the **side friction factor.** This term also depicts the lateral acceleration acting on a vehicle which is the product of the side friction demand factor and the gravitational constant. Vehicle speeds on horizontal curves create tire side thrust which is offset by the frictional forces between the tires and the riding surface.

AASHTO's "simplified curve formula" (shown below) is a basic side friction equation that produces slightly higher friction estimates than those resulting from the "basic curve formula".

$$f = \frac{V^2}{15R} - 0.01\ e$$

f = side friction factor (demand)

e = rate of roadway superelevation (percent)

V = vehicle speed (mph)

R = radius of curve (feet)

The point of impending skid is the upper side friction factor limit where the tires begin to skid. This depends on vehicle speed, road surface type/condition, and tire condition/type. Historical data has shown a decrease in friction as vehicle speeds increase. Since roadway curves are designed with a margin of safety to prevent skidding, the design friction values should be substantially less than impending skid values.

Maximum side friction factors should be conservative for dry conditions with an ample margin of safety against skidding on wet or icy pavements or vehicle rollover. This shows the need for using skid-resistant surfacing due to roadway friction demands from driving maneuvers (braking, lane changes, directional changes, etc.). Recent studies confirm that side friction factors need to be lower for high-speed designs versus low-speed ones.

Side Friction Design Factors

	Speed (mph)	Side Friction Factor (f)
Low-Speed Design	10	0.38
	20	0.26
	30	0.20
	40	0.17
High-Speed Design	50	0.14
	60	0.12
	70	0.10
	80	0.08

Figure 3-6 from AASHTO's "Green Book" shows the recommended side friction factors for horizontal curve design with maximum values ranging from 0.14 (50 mph) to 0.08 (80 mph).

The level of lateral acceleration that causes drivers to avoid higher speeds is the key to selecting maximum side friction factors.

NORMAL CROSS SLOPE

Roadway drainage determines the minimum rate of cross slope for the traveled way. Acceptable minimum cross slope values range from 1.5 to 2.0 percent (with 2.0 typically used for paved, uncurbed pavements) depending on the roadway type and weather conditions.

MAXIMUM SUPERELEVATION RATES

No single maximum superelevation rate is universally applicable. In order to promote design consistency, a maximum rate is desirable for locations with similar characteristics (land usage, climate, etc.). This uniformity encompasses the roadway's alignment as well as its associated design elements and driver expectations. Consistent designs are associated with lower workloads and crash frequencies.

Controls for Maximum Superelevation

Climate (amount of precipitation)
Terrain (flat, rolling, or mountainous)
Area type (rural or urban)
Slow-moving vehicles (frequency)

Eight percent (8%) is considered to be a reasonable maximum superelevation rate. The highest superelevation rate for highways is typically 10 percent - rates greater than 12% are considered beyond practical limits but may be used in some cases (i.e. low-volume gravel roads for cross drainage).

Recommendations

- Several maximum superelevation rates should be used for design controls for horizontal curves
- Do not exceed a rate of 12 percent
- Rates of 4 or 6 percent may be used for urban areas with few constraints
- Superelevation may be omitted on low-speed urban roads with severe constraints

MINIMUM CURVATURE

The minimum radius for horizontal curves is a limiting value for design speeds based on the maximum superelevation and maximum side friction factor. Actual design values were developed from the laws of mechanics and depend on practical limits and factors that were determined empirically. Sharper radii would require superelevation above the limits for comfortable operation. The minimum radius values maintain a margin of safety against vehicle rollover and skidding.

The "basic curve equation" governs vehicle operation on a horizontal curve.

$$\frac{0.01e+f}{1-0.01ef}=\frac{v^2}{gR}=\frac{0.067V^2}{R}=\frac{V^2}{15R}$$

f = side friction factor (demand)

e = rate of roadway superelevation (percent)

v = vehicle speed (feet/second)

g = gravitational constant (32.2 ft/sec²)

V = vehicle speed (mph)

R = radius of curve (feet)

The following equation can be used to calculate the minimum radius of curvature, R_{min} from the "simplified curve formula".

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

f_{max} = maximum side friction factor (demand)

e_{max} = maximum rate of roadway superelevation (percent)

V = vehicle speed (mph)

R = radius of curve (feet)

Horizontal curve equations utilize a radius measured to vehicle center of gravity (center of inner travel lane). These equations neglect roadway width or horizontal control location. The difference between the centerline and center of gravity is minor for two-lane roadways – so this curve radius should be measured to the road's centerline.

GRADES

Motorists typically drive faster on downgrades versus upgrades for long or steep roadway grades. Data has shown greater side friction demands on

downgrades – due to braking forces
and **steep upgrades** – from tractive forces.

For grades steeper than 5 percent, adjusting superelevation rates may be considered since this is crucial to roadways with heavy truck volume or intermediate curves with high levels of side friction. This adjustment may be done without reducing the design speed for the upgrade. The proper speed variation depends on specific conditions (grade rate, length, curve radius, etc.) compared to other curves on the roadway's approaches.

Additional superelevation for upgrades on two-lane and multilane undivided roads can counter side friction loss due to tractive forces. This addition on long upgrades may cause negative side friction for slow moving vehicles (heavy trucks, etc.) but may be alleviated by slower speeds, more time for counter steering, and increased driver experience/training.

For rural highways, urban freeways, and high speed urban streets, a balanced design of superelevated, successive horizontal curves is desired to provide a smooth transition with maximum side friction factors varying from 0.14 (50 mph) to 0.08 (80 mph).

On low-speed urban streets, superelevation on horizontal curves may be minimized or eliminated with lateral forces being sustained by side friction only. Various factors that may make superelevation unsuited for low-speed urban areas include:

- Wide pavement areas
- Need to meet adjacent property grades
- Surface drainage
- Low-speed operation concerns
- Intersection frequency

TURNING ROADWAYS

Turning roadways include interchanges (loop or diamond configurations with tangents and curves) and intersections (diamond configurations with compound curves) for right-turning vehicles.

The minimum radii for right-turning vehicles on turning roads must be measured from the inner edge of the traveled way. The radius and superelevation are determined from design speed and other values. Sharper curves with shorter lengths have a reduced opportunity for larger superelevation rates. The desirable turning speed is the average running speed of traffic approaching the turn. Maximum superelevation values should be used on ramps to prevent skidding/overturning, when possible.

Compound curves can be used exclusively for turning roadways with design speeds of 45 mph or less. Higher design speeds make their use impractical due to the large amounts of right-of-way required and should include a mixture of tangents and curves.

TRANSITION DESIGN CONTROLS

A number of factors determine horizontal curve safety, including

curve length *radius* *spiral transitions* *roadway superelevation.*

Since roadway crashes are more probable at curves with small radii or insufficient superelevation, spiral transitions may be used to decrease these mishaps.

Horizontal alignment transition section designs include:

Superelevation transition

- transitions in the roadway cross slope
- consists of superelevation runoff section for outside-lane cross slope changes (flat to full superelevation); and tangent runout section (normal to flat)

Alignment transition

- transitional curves in the horizontal alignment
- spiral or compound curve may be used
- produces gradual change in roadway curvature

When both transition sections are used, these are integrated at the beginning and end of the mainline circular curves.

There is no standard accepted empirical basis for determining runoff lengths. Superelevation runoff lengths are mainly governed by appearance. Control runoff lengths (100 to 650 ft range) are commonly determined as a function of the slope of the outside edge of the traveled way relative to the roadway centerline profile.

TANGENT-TO-CURVE TRANSITIONS

"Tangent-to-curve" transitions are used for locations where roadway tangents directly adjoin the main circular curve - without using transition curves. **Superelevation runoff** is the length of roadway needed to transition the lane cross slope from flat to full superelevation and conversely. This length should be based on a maximum acceptable difference between the longitudinal grades of the axis of rotation (alignment centerline or pavement reference lines) and the pavement edge. The grade difference (relative gradient) should be limited to a maximum value of 0.50 percent or a longitudinal slope of 1:200 at 50 mph. Greater slopes may be used for design speeds less than 50 mph.

Maximum relative gradients vary with design speed to provide shorter runoff lengths at lower speed and longer lengths at higher speeds. Relative gradient values of 0.78 and 0.35 percent have been shown to provide adequate runoff lengths for 15 and 80 mph.

Maximum Relative Gradients

Design Speed (mph)	Maximum Relative Gradient (%)	Equivalent Maximum Relative Slope
20	0.74	1:135
30	0.66	1:152
40	0.58	1:172
50	0.50	1:200
60	0.45	1:222
70	0.40	1:250
80	0.35	1:286

Source: AASHTO "Green Book" Table 3-15

MINIMUM LENGTH OF SUPERELEVATION RUNOFF

The AASHTO equation for determining the minimum length of superelevation runoff is based on design speed, superelevation, and roadway width. This equation can be used for rotation about any pavement reference line containing a rotated width (wn1) with a common rate of superelevation and rotated as a plane.

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

L_r = minimum length of superelevation runoff (ft)

n_l = number of lanes rotated

e_d = design rate of roadway superelevation (percent)

b_w = adjustment factor for number of lanes rotated

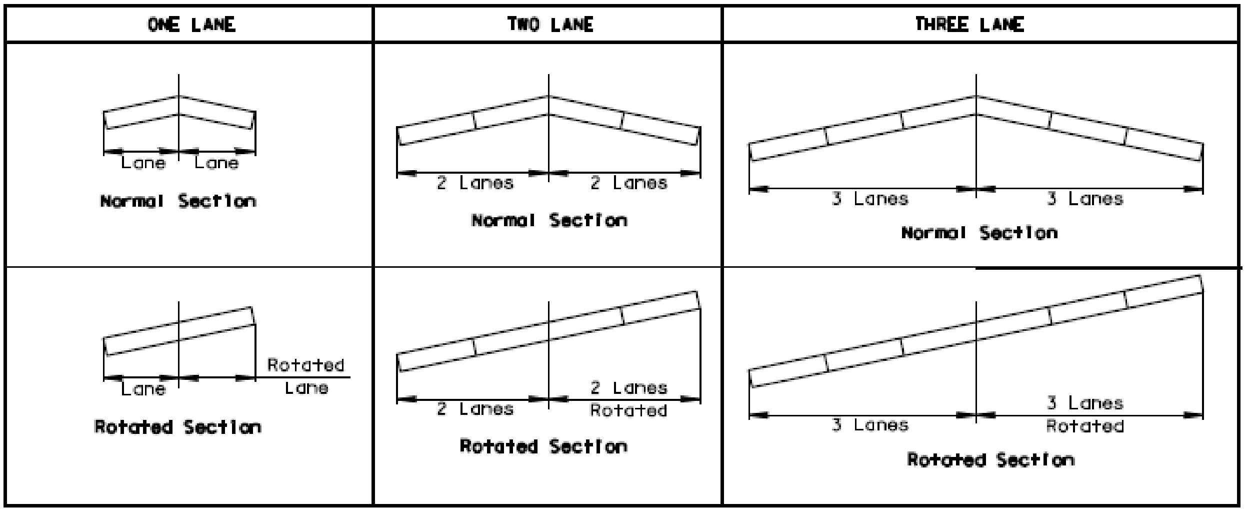
w = width of one traffic lane (feet)

Δ = maximum relative gradient (percent)

Adjustment Factor - No. of Lanes Rotated

Number of Lanes Rotated n_1	Adjustment Factor b_w	Length Increase Relative to One-Lane Rotated $n_1 b_w$
1	1.00	1.00
1.5	0.83	1.25
2	0.75	1.50
2.5	0.70	1.75
3	0.67	2.00
3.5	0.64	2.25

Source: AASHTO “Green Book” Table 3-16



Tangent Runout Length Factors

- Amount of adverse cross slope to be removed
- Rate of removal

The removal rate needs to equal the relative gradient that defined the superelevation runoff length in order to produce a smooth edge of pavement profile.

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

L_t = minimum length of tangent runoff (feet)

e_{NC} = normal cross slope rate (percent)

e_d = design rate of roadway superelevation (percent)

L_r = minimum length of superelevation runoff (feet)

LOCATION WITH RESPECT TO END OF CURVE

Locating a curve's superelevation runoff with respect to its Point of Curvature (PC) is an important ingredient for tangent-to-curve design. The preferable method uses a portion on the tangent since it minimizes peak lateral acceleration and side friction demand, plus it is consistent with the natural spiral path during curve entry. A typical superelevation runoff length is divided between the tangent and curve sections (avoiding placement of the entire length in either section). The tangent proportion normally varies from 60 to 80 percent – with most entities using 67 percent. Theoretical factors indicate that tangent runoff length values of 70 to 90 percent produce the best operating conditions with the specific value depending on design speed and rotated width.

Runoff Locations that Minimize Lateral Motion

Design Speed (mph)	Runoff Located Prior to Curve No. of Lanes Rotated			
	1.0	1.5	2.0 to 2.5	3.0 to 3.5
15 to 45	0.80	0.85	0.90	0.90
50 to 80	0.70	0.75	0.80	0.85

Source: AASHTO "Green Book" Table 3-18

SPIRAL CURVES

The average driver can follow a suitable transition path when entering or exiting a circular horizontal curve and stay within normal lane limits. At locations with high speeds and sharp curvature, the use of transition curves between the tangents and the curves may make it easier for the vehicle to stay within its own lane.

Spiral curves are typically incorporated into horizontal alignments to transition from normal tangent sections to full superelevation. Spiral radii decrease uniformly from infinity (at the tangent) to that of the adjoining curve. By being more complex, spirals provide excellent operational capabilities – especially for high speed alignments.

Uses of Spiral Transition Curves

- Tangent with a circular curve
- Tangent with a tangent (double spiral)
- Circular curve with a circular curve
  and compound or reverse curves

Advantages of Transition Curves

Natural easy-to-follow driving

- *Lateral force increases and decreases gradually*
- *Minimizes adjoining lane encroachment*
- *Promotes uniform speeds*

Suitable location for superelevation runoff

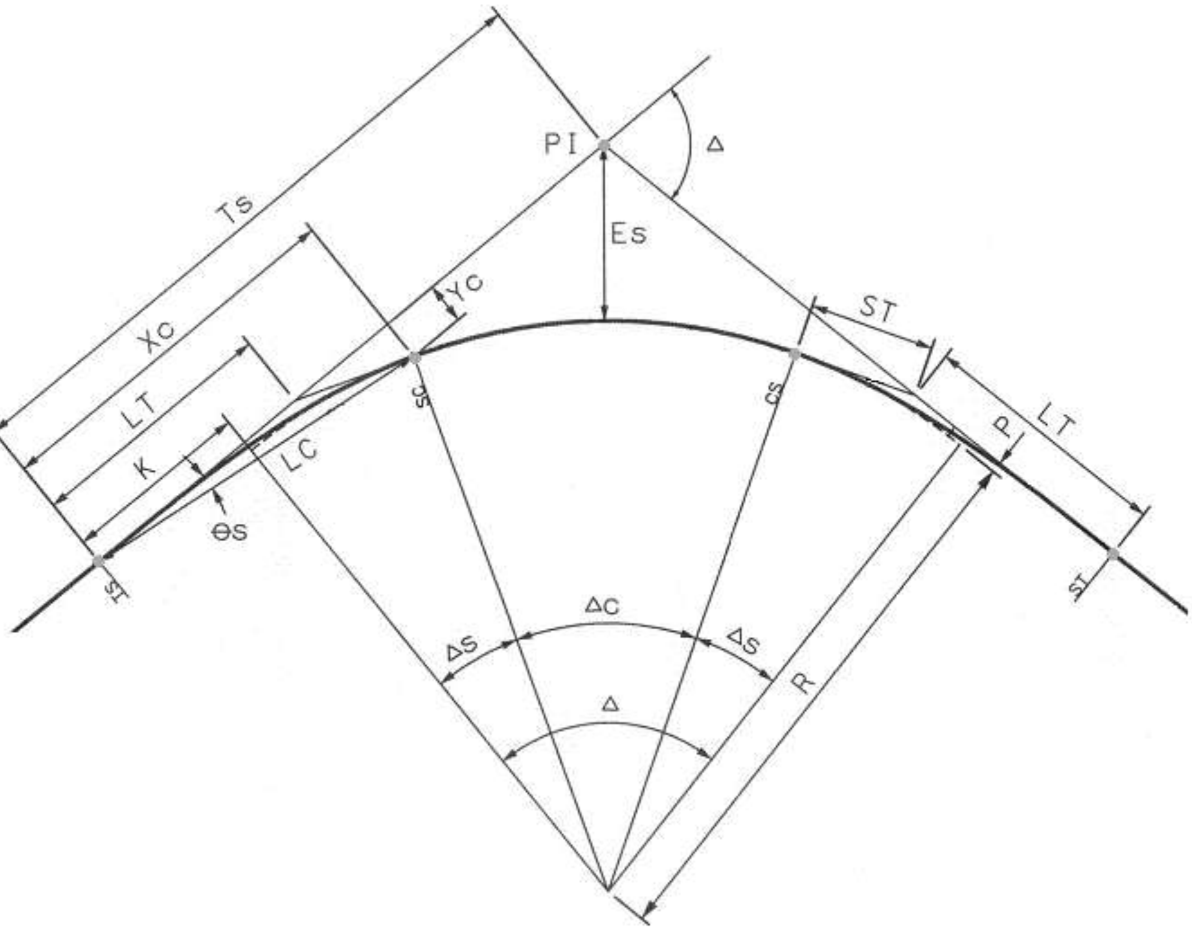
- *Fits speed-radius relationship for vehicles*

Facilitates traveled way width transition

- *Provides flexibility for width transitions on sharp circular curves*

Enhances roadway appearance

- *Avoids perceived breaks in the horizontal alignment*



LENGTH OF SPIRAL

In 1909, W.H. Short developed an equation using lateral acceleration on railroad curves. This basic equation is used by some agencies for calculating the minimum length of a spiral curve. The minimum length of a spiral curve may be determined from the following AASHTO formula.

$$L = \frac{3.15V^3}{RC}$$

L = minimum length of spiral (feet)

C = rate of increase of lateral acceleration (ft/sec³)

V = vehicle speed (mph)

R = radius of curve (feet)

The *C*-factor represents comfort and safety levels and normally ranges from 1 to 3 ft/sec^3 for highways. Equations modified for superelevation produce shorter spiral curve lengths. AASHTO states that a more realistic method is to set the spiral length equal to the superelevation runoff.

MAXIMUM SPIRAL RADIUS

Present guidance regarding spiral curves indicates that an upper radius limit can be used – with radii below this value having safety and operational benefits from using spirals. Minimum lateral acceleration rates of 1.3 to 4.25 ft/s^2 have been used to establish limiting radii. The higher rates correspond to the maximum radius with a reduction in crash potential. AASHTO recommends that the maximum spiral radius should be based on a minimum lateral acceleration rate of 4.25 ft/s^2. This produces a range of values (Table 3-20) such as:

Design Speed	Maximum Radius
15 mph	114 feet
to	to
80 mph	3238 feet

MINIMUM SPIRAL LENGTH

Spiral curve length is a crucial design control for horizontal alignments. *Driver comfort* and *lateral vehicle shift* are the major considerations used to define the minimum length of spiral curve. Criteria that address driver comfort help produce an easy increase in lateral acceleration upon spiral curve entry. Considerations for lateral shifting are meant to create a spiral that can handle a shift in vehicle lateral position within the travel lane that is consistent with its natural spiral path. AASHTO Equations 3-26 and 3-27 illustrate these relationships.

$L_{s,min}$ should be be the larger value of

$$L_{s,min} = \sqrt{24(p_{min})R}$$

or

$$L_{s,min} = \frac{3.15V^3}{RC}$$

$L_{s,min}$ = minimum length of spiral (feet)

C = maximum rate of change in lateral acceleration (4 ft/sec^2)

V = vehicle speed (mph)

R = radius of curve (feet)

p_{min} = minimum lateral offset between the tangent and circular curve (0.66 ft)

The standard value for p_{min} is typical for natural steering behavior. Using lower values for C and p_{min} will create longer, easier spiral lengths that will not exhibit the minimum lengths associated with driver comfort.

MAXIMUM LENGTH OF SPIRAL

A conservative maximum length of spiral transition curve needs to be determined in order to prevent violating driver expectations about the sharpness of upcoming curves. AASHTO Equation 3-28 produces appropriate values for maximum spiral lengths with the following formula:

$$L_{s,max} = \sqrt{24(p_{max})R}$$

$L_{s,max}$ = maximum length of spiral (feet)

R = radius of circular curve (feet)

p_{max} = maximum lateral offset between the tangent and circular curve (3.3 feet)

The recommended p_{max} value of 3.3 feet is consistent with the maximum lateral shift plus it balances spiral length and curve radius values.

DESIRABLE LENGTH OF SPIRAL

Research has proven that optimal conditions occur when spiral curve lengths are equal to the natural spiral path lengths of vehicles. Length differences produced operational problems involving large lateral velocities or shifts at the end of the transition. AASHTO Table 3-21 provides a table of desirable lengths of spiral that correspond to 2.0 seconds of travel time (for natural spiral paths). If the desirable spiral value is less than the calculated minimum spiral curve length – use the minimum length for design.

Design Speed	Spiral Length
15 mph	44 feet
to	to
80 mph	235 feet

LENGTH OF SUPERELEVATION RUNOFF

While it is recommended that superelevation runoffs occur over the spiral length, calculated runoff lengths and lengths of spiral are not significantly different. Lengths of superelevation runoff apply to superelevated curves and are recommended when determining minimum spiral lengths – the spiral length should be set equal to the runoff length. By transitioning the superelevation over the spiral length, full superelevation is contained within the whole circular curve. However, a result of equating the runoff and spiral lengths is a resulting relative gradient that exceeds AASHTO's maximum relative gradients.

LENGTH OF TANGENT RUNOFF

Tangent runout lengths for spirals are akin to the designs for tangent-to-curve transitions. The preferred design contains a smooth pavement edge profile with a common edge slope gradient throughout the superelevation runout and runoff sections. AASHTO Equation 3-29 presents a computation method for tangent runout lengths. The "Green Book" also provides a table (Table 3-23) for tangent runout lengths.

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

L_t = length of tangent runoff (feet)

e_{NC} = normal cross slope rate (percent)

e_d = design rate of roadway superelevation (percent)

L_S = length of spiral curve (feet)

LOCATION WITH RESPECT TO END OF CURVE

The superelevation runoff is accomplished over the whole spiral transition and should be equal for both the tangent-to-spiral (TS) and spiral-to-curve (SC) transitions. The spiral curve and superelevation runoff are equivalent with the roadway rotated to full superelevation at the SC, and reversed when leaving the curve. The whole circular curve contains the full superelevation.

METHODS OF ATTAINING SUPERELEVATION

*Revolving traveled way with normal cross slopes about **centerline** profile*

Most widely used method due to its reduced distortion involving a change in elevation of the edge of the traveled way. One-half of the elevation change is made at each edge.

*Revolving traveled way with normal cross slopes about **inside-edge** profile*

The inside-edge profile is parallel to the profile reference line. The actual centerline profile is raised with respect to the inside-edge profile to create one-half of the elevation change. The other half is made by raising the outside edge profile an equal amount (with respect to actual centerline).

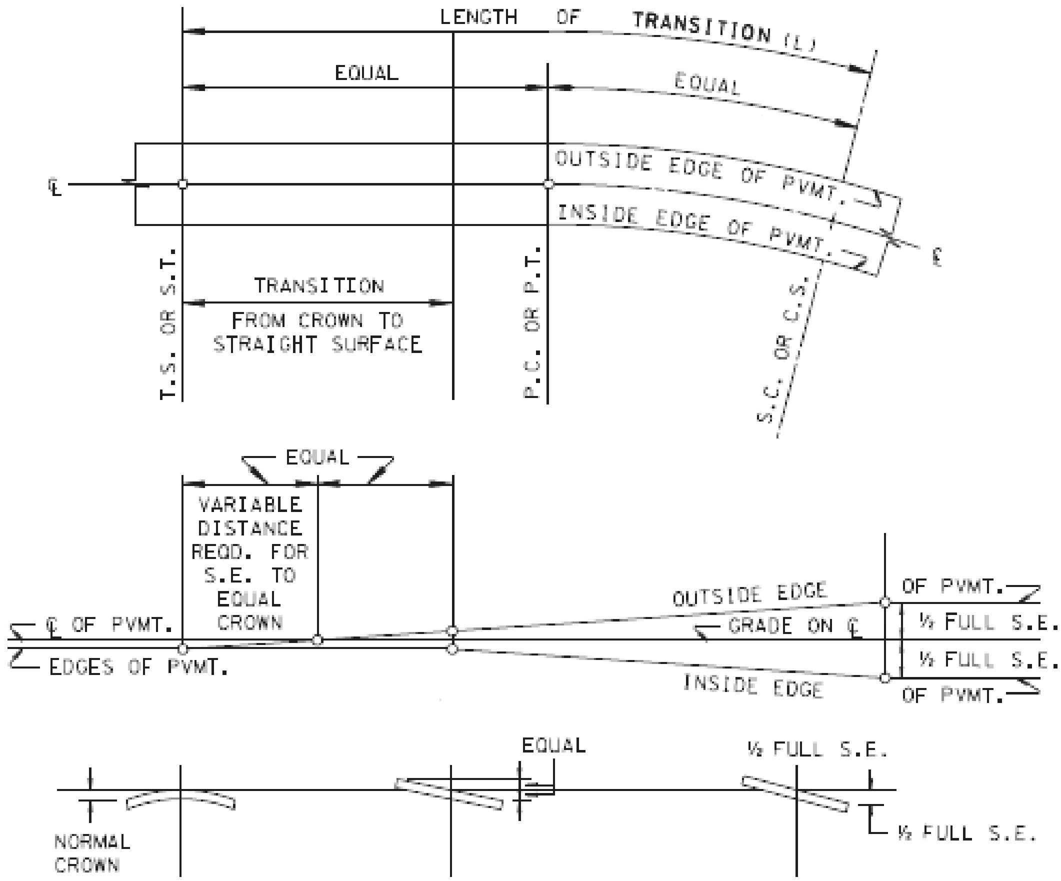
*Revolving traveled way with normal cross slopes about **outside-edge** profile*

Similar to inside-edge method except the elevation change occurs below the outside-edge profile.

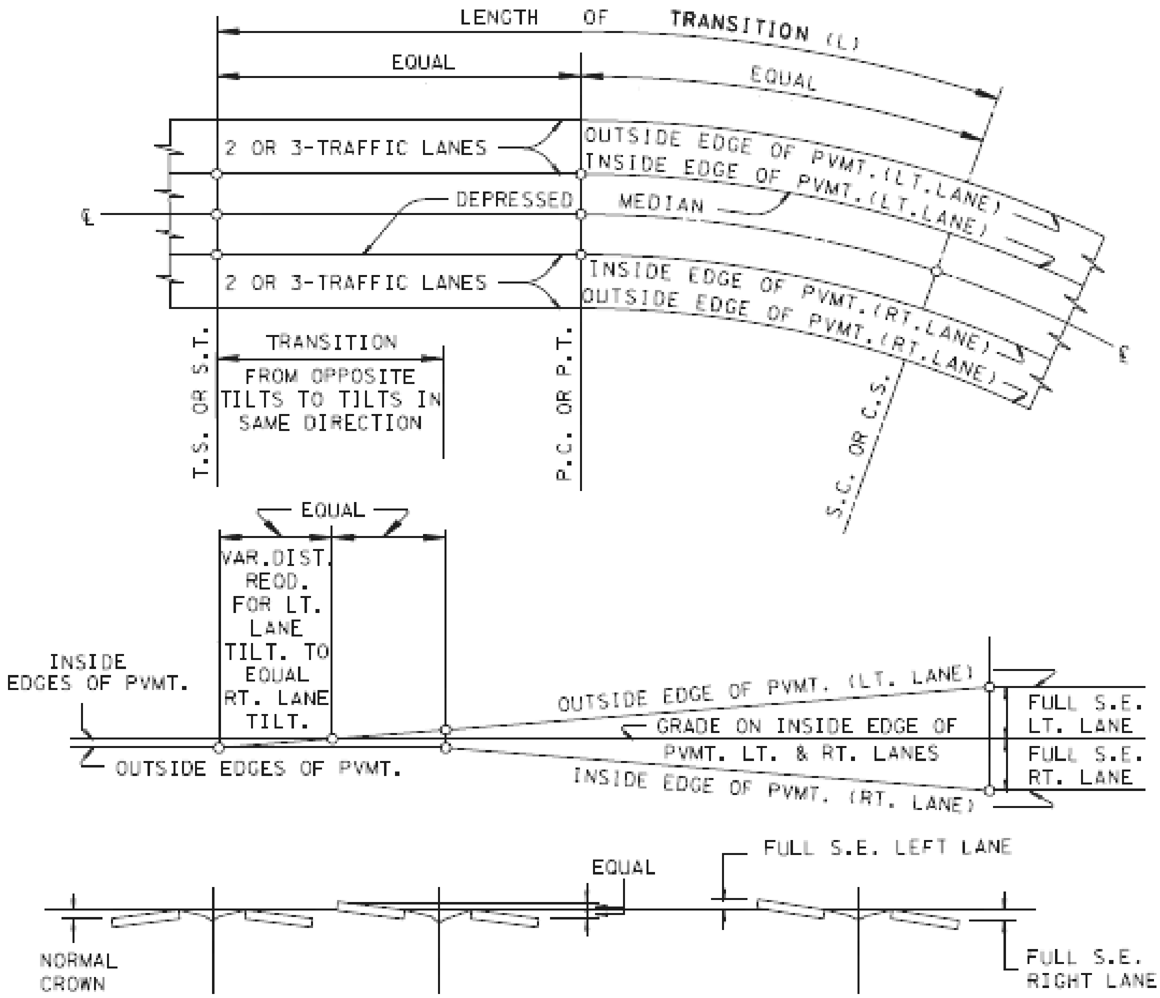
*Revolving traveled way with **straight** cross slopes about outside-edge profile*

Often used for two-lane one-way roads where the axis of rotation coincides with the edge of traveled way adjacent to the median.

The centerline profile's shape and direction can influence the method for attaining superelevation. Each transition section should be evaluated individually to produce the most pleasing and functional results. The following figures illustrate these different superelevation methods for a curve veering to the right.



Typical Superelevation Transition
2-Lane Roadway



Typical Superelevation Transition
Roadway With Depressed Median

DESIGN OF SMOOTH PROFILES FOR TRAVELED-WAY EDGES

Vertical curves should be used to smoothe out any angular breaks at cross sections caused by the tangent profile control lines. Presently, there are no specific guidelines for vertical curve lengths in diagrammatic profiles but the minimum vertical curve length can be approximated using 0.2 times the design speed (greater lengths can be used where practical).

Another method uses graphical techniques (spline-line development) to define the edge profile. The profile is first plotted on a vertical scale with superelevation control points. Next, curves or templates are used to approximate straight-line controls. After smoothing is completed, elevations can then be read properly. This method offers infinite alternatives with minimal labor.

Divided roadways need greater emphasis in their design and appearance due to their heavier traffic volumes. Therefore, the use of smooth profiles for divided highway traveled-way edges is warranted more than those for two-lane roads.

AXIS OF ROTATION WITH A MEDIAN

The addition of a roadway median impacts superelevation transition designs due to the location for the axis of rotation. These locations are dependent on median width and cross-section which are described in the following common combinations:

Case I - *Traveled way (with median) superelevated as a plane section*

Limited to narrow medians and moderate superelevation to avoid major elevation differences (edges of traveled way)
Median width: 15 feet or less
Length of runoff is based on total rotated width (including median)
Median widths 10 feet or less may be deleted from runoff length since narrow medians have little effect

Case II - *Median as a horizontal plane with the two traveled ways rotated separately about median edges*

Suitable median widths: 15 to 60 feet
Usually used for roadways rotated about median-edge of pavement
Medians widths 10 feet or less may have runoff lengths the same as single undivided roads

Case III - *Two traveled ways treated separate for runoff*

Produces variable difference in median edge elevations
Wide median widths: 60 feet or more
Profiles and superelevation transition designed separately for two roadways

MINIMUM TRANSITION GRADES

There are two types of drainage problems for pavement surfaces in superelevation transition sections.

1) Potential lack of adequate longitudinal grade
   Grade axis of rotation is equal (but opposite sign) to effective relative gradient
   Results in pavement edge with negligible longitudinal grade and poor surface drainage

2) Inadequate lateral drainage
   Due to negligible cross slope during pavement rotation
   Length of transition includes tangent runout and equal runoff sections that may not drain pavement laterally

Potential drainage problems may be alleviated using the following techniques:

- Maintaining a minimum vertical grade of 0.5% through the transition
- Maintaining minimum edge-of-pavement grades of 0.2% (0.5% curbed streets) through the transition

TURNING ROADWAY CURVES

Drivers naturally follow transitional travel paths when turning at interchange ramps and intersections (or on open roadways). Facilities not following natural transition paths may result in drivers deviating from the intended path and encroaching on other traffic lanes. The best ways to accommodate natural travel paths is by using transition curves - either between two circular arcs, or between a tangent and circular curve.

Spiral lengths for intersection curves are determined using the same method as for open roadways. Intersection curve lengths may be less than highway curves since motorists accept quicker changes in travel direction at intersections.

The minimum spiral lengths for minimum-radius curves are determined by design speed. AASHTO Table 3-24 shows values ranging from:

Design Speed	Design Minimum Speed Length
20 mph	70 feet
to	to
45 mph	200 feet

COMPOUND CIRCULAR CURVES

Compound circular curves can be used to produce effective turning roadway geometries for intersection and interchanges. For locations where circular arcs with different radii are connected, the following ratios are generally acceptable:

Compound Curve Location	Flatter Radius to Sharper Radius Ratio
Open highways	1.5:1
Intersections/Turning roadways	2:1 (satisfactory operation & appearance)

Smaller curve radii differential is preferred where practical – with a desirable maximum value of 1.75:1. If the ratio is greater than 2:1, a suitable intermediate spiral/arc should be used between the two curves. Do not use this ratio control for very sharp curves designed for minimum vehicle turning paths. Higher ratios may be needed for compound curves that closely fit the design vehicle path. Each curve length should be adequate for reasonable driver deceleration.

AASHTO Table 3-25 provides circular arc lengths for compound intersection curves. These values assumed a deceleration rate of 3 mph/s with a desirable minimum deceleration of 2 mph/s (very light braking).

OFFTRACKING

Offtracking occurs when a vehicle's rear wheels do not follow the exact path as its front wheels when negotiating a horizontal curve or turn. This is dependent on curve/turning radii, articulation points, and vehicle wheelbase lengths.

Situation	Result
Curve without superelevation (low speed)	Rear wheels track inside front wheels
Superelevated curve	Rear wheels may track inside front wheels (more or less)
High speeds	Rear wheels may track outside front wheels

Offtracking is more pronounced for larger design vehicles and emphasizes the amount of widening needed on horizontal curves. This widening increases with the size of the design vehicle and decreases with increasing curve radii.

The amount of widening on horizontal curves for offtracking depends on curve radius design vehicle characteristics:

Width of inner lane vehicle front overhang
Rear overhang width
Track width for passing
Lateral vehicle clearance
Curve difficulty allowance width

TRAVELED-WAY WIDENING ON HORIZONTAL CURVES

Often, traveled ways on horizontal curves may need to be widened to produce operational characteristics that are similar to tangent sections. While the need for widening on modern highways is less than that for past roadways, there are some cases where *speed, curvature, or width* may require appropriate traveled way widening.

Primary Reasons for Widening on Curves

- Design vehicle off tracks when negotiating curve
- Driver difficulty in remaining in center of the lane

AASHTO Equation 3-34 calculates the amount of traveled way widening for horizontal curves by using the difference between the width needed on the curve and the tangent width. The needed curve width has several variables: track width for passing/meeting vehicles; lateral vehicle clearance; width of inner lane vehicle front overhang; curve difficulty allowance width.

In most cases, the design vehicle is a truck (typically WB-62) since off tracking is much greater for heavy vehicles versus passenger cars.

Since sight distance can be restricted when meeting opposing vehicles on curved two-way roads, widening procedures for two-lane, one-way traveled way (divided highway) should be similar to those for two-lane, two-way roadways.

Any widening for horizontal curves should transition gradually on the approaches to align traveled way edges and vehicle paths. AASHTO Equation 3-35 provides values for the width of traveled way on curves.

Curve Widening Design Concerns

- Widen only the inside edge of traveled way for simple curves
- Widen on the inside edge or equally divided from the centerline for spirals
- Transition gradually over the length (typically 100 to 200 ft) to make all traveled way fully usable
- Avoid tangent transition edges – no angular breaks at pavement edges
- One-half to two-thirds transition length should be along tangent sections for roads without spirals

- Width increases should be distributed along the spiral length for highways with spirals
- Fully detail widening areas on construction plans

Factors for determining turning roadway widths at intersections include:

- Expected speed
- Curve radius: combined with design vehicle track width determine turning roadway width
- Types of vehicles: size and frequency of users or expected users

Turning roadways are classified operationally as:

one-lane (with or without passing opportunities)

or *two-lane (one-way or two-way)*

<u>Design Methods for Turning Roadways</u>

Case I
One-lane, one-way operation
No passing stalled vehicles provision
For minor turning movements, moderate turning volumes, short connecting roadway
Remote chance of vehicle breakdown
Preferable sloping curb or flush edge of traveled way

Case II
One-lane, one-way operation
Contains passing provision for stalled vehicles
Low speeds with adequate passing clearance
Sufficient widths for all turning movements of moderate to heavy traffic volumes within capacity of single-lane connection
For breakdowns, low traffic can be maintained

Case III
Two-lane operation, either one or two-way
Two lanes necessary for traffic volume

Since precise data regarding traffic volumes for each vehicle type is not readily available, traffic conditions used to define turning roadway widths are described in broad terms.

Traffic Conditions for Turning Roadway Widths

Traffic Condition A	Predominantly Passenger Car (P) vehicles Some Single-Unit Trucks (SU-30) Small volume of trucks with occasional large truck
Traffic Condition B	Majority of Single-Unit Trucks (SU-30) Some tractor- semitrailer combination trucks (WB-40) Moderate volume of trucks - 5 to 10%
Traffic Condition C	Predominantly tractor-semitrailer combo (WB-40) More and larger trucks

Widths for turning roadways include shoulders or lateral clearance outside the traveled way. Shoulder widths may vary from none (curbed urban streets) to open-highway cross-section.

Usable Shoulder Widths or Lateral Clearances Outside of Turning Roadways

Turning Roadway Condition	Shoulder Width or Lateral Clearance	
	Left	Right
Short length and/or channelized intersection	2 to 4 ft	2 to 4 ft
Intermediate to long length or in cut/fill section	4 to 10 ft	6 to 12 ft

Source: AASHTO "Green Book" Table 3-30

For roadways without curbs or with sloping curbs, adjacent shoulders should match the type and cross section of the approaches.

If roadside barriers are present, shoulder widths should be measured to the face of barrier with a graded width of 2.0 feet.

For other than low-volume roadways, right shoulders should be stabilized a minimum of 4.0 feet.

SIGHT DISTANCE

Sight distance is the length or distance of roadway visible to the driver. This is a major design control for vertical alignments and is essential for the safe and efficient operation of vehicles. This distance is dependent on the driver's eye height, the specified object height, and the height/position of sight obstructions. The three-dimensional features of the roadway should provide a minimum sight line for safe operations.

Sight Distance Criteria

Height of Driver's Eye:	3.50 feet above road surface (passenger vehicles)
	7.60 feet above road surface (trucks)
Height of Object:	2.00 feet above road surface (stopping & decision)
	3.50 feet above road surface (passing & intersection)

Due to differences in driver needs, various types of sight distance apply to geometric design **Stopping,** **Decision,** **Passing,** and **Intersection**.

STOPPING SIGHT DISTANCE (SSD)

Stopping sight distance is considered to be the most basic form of sight distance. This distance is the length of roadway needed for a vehicle traveling at design speed to stop before reaching a stationary object in the road. Ideally, all of the roadway should provide stopping sight distance consistent with its design speed. However, this distance can be affected by both horizontal and vertical geometric features.

Stopping sight distance is composed of two distances:

(1) Brake Reaction Time starts upon driver recognition of a roadway obstacle until application of the vehicle's brakes. Typically, the driver not only needs to see the object but also recognize it as a potential hazard. The time required to make this determination can widely vary based on the object's distance, visibility, roadway conditions, vehicle speed, type of obstacle, etc.

Perception →Braking

From various studies, it was shown that the required brake reaction time needed to be long enough to encompass the majority of driver reaction times under most roadway conditions. A brake reaction time of 2.5 seconds met the capabilities of most drivers – including older drivers.

The recommended brake reaction time of **2.5 seconds** exceeds the 90th percentile of driver reaction time and is considered adequate for typical roadway conditions – but not for most complex driving conditions that may be encountered.

(2) Braking Distance – Roadway distance traveled by a vehicle during braking (from the instant of brake application)

Braking →Stopping

Using the following equation, the approximate braking distance (d_B) may be calculated for a vehicle traveling at design speed on a level roadway. The recommended deceleration rate (a) of **11.2 ft/s²** has shown to be suitable since 90% of all drivers decelerate at greater values. This deceleration rate is fairly comfortable and allows drivers to maintain steering control.

$$d_B = 1.075\frac{V^2}{a}$$

d_B = braking distance (feet)

V = design speed (mph)

a = deceleration rate (ft/sec^2)

For roadways on a grade, the braking distance can be determined by:

$$d_B = \frac{V^2}{30\left[\left(\frac{a}{32.2}\right) +/- G\right]}$$

d_B = braking distance on grade (feet)

V = design speed (mph)

a = deceleration rate (ft/sec^2)

G = grade (ft/ft)

Stopping distances for downgrades are longer than those needed for level roads while those on upgrades are shorter.

The Stopping Sight Distance formula is a function of initial speed, braking friction, perception/reaction time, and roadway grade that contains assumptions about the driver's eye height (3.5 feet) and the size of object in the road (2 feet).

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

SSD = stopping sight distance (feet)

V = design speed (mph)

a = deceleration rate (ft/sec^2)

Stopping Sight Distance – Level Roadways

Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
20	73.5	38.4	111.9	115
30	110.3	86.4	196.7	200
40	147.0	153.6	300.6	305
50	183.8	240.0	423.8	425
60	220.5	345.5	566.0	570
70	257.3	470.3	727.6	730
80	294.0	614.3	908.3	910

Source: AASHTO "Green Book" Table 3-1

Limitations of the AASHTO Model

- Does not fully account for heavy vehicles (longer stopping times)
- Does not differentiate between various highway types
- Does not recognize differing roadway conditions

Proper roadway design should address these variables by providing more than minimum stopping sight distance at locations with vehicle conflicts or hazardous conditions (sharp curves, cross-section changes, intersections, etc.).

DECISION SIGHT DISTANCE (DSD)

Certain situations requiring complex decisions or maneuvers (unexpected conflicts, navigational needs, roadway changes, etc.) can place extra demands on drivers. These circumstances usually require longer sight distances than those for stopping.

Decision sight distance recognizes these needs and is composed of the following required actions:

Detect unexpected/unusual conflict
Recognize potential risk
Select appropriate speed /path
Initiate and complete safe maneuver

Decision sight distance values are substantially greater than those for Stopping Sight Distance since DSD provides an additional margin of error and sufficient maneuver length at vehicle speeds – *rather than just stopping*.

Decision sight distance is needed for a variety of roadway environments – such as bridges, alignment changes, interchanges, intersections, lane drops, congested intersections, median crossovers, roadway cross-section changes, toll facilities, and unusual geometric configurations.

DSD values depend on whether the roadway's location is rural or urban, and the type of avoidance maneuver required.

Avoidance Maneuver	Condition	Time (sec)
A	Stop on rural road	3.0
B	Stop on urban road	9.1
C	Change on rural road	10.2 to 11.2
D	Change on suburban road	12.1 to 12.9
E	Change on urban road	14.0 to 14.5

The "Green Book" provides tabular decision sight distances to provide appropriate values for critical locations, and to furnish suitable evaluation criteria of available sight distances. Critical decision points need to have sufficient DSD.

Decision Sight Distance

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

Source: AASHTO "Green Book" Table 3-3

The pre-maneuver time for avoidance maneuvers is greater than the brake reaction time for Stopping Sight Distance. This gives drivers extra time to recognize the situation, identify alternatives, and initiate a response. DSD pre-maneuver components typically range from **3.0 to 9.1 seconds.**

For Avoidance Maneuvers A and B, the braking distance (for design speed) was added to the pre-maneuver component. Decision sight distances for **Avoidance Maneuvers A and B** can be calculated using the following formula:

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

DSD = decision sight distance (feet)

V = design speed (mph)

a = driver deceleration rate (ft/sec^2)

t = pre-maneuver time (seconds)

For Avoidance Maneuvers C thru E, the braking component is replaced with maneuver distance based on times (3.5 to 4.5 seconds) that decrease with increasing speed. Decision sight distances for **Avoidance Maneuvers C, D, and E** can be calculated from the following equation:

$$DSD = 1.47Vt$$

DSD = decision sight distance (feet)

V = design speed (mph)

t = total pre-maneuver and maneuver time (seconds)

PASSING SIGHT DISTANCE

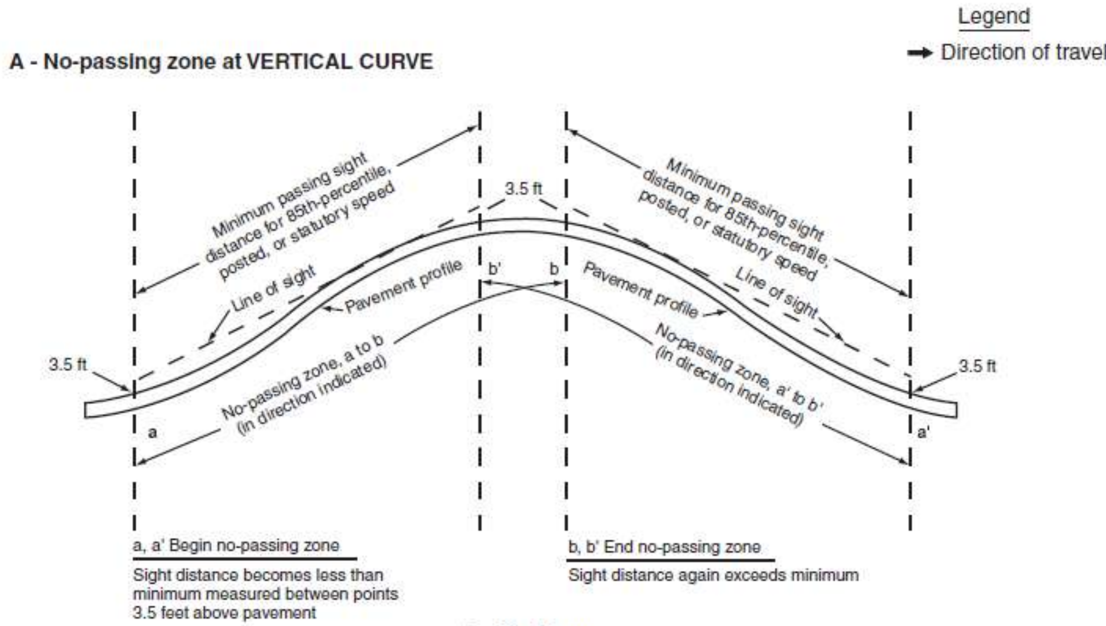
Passing sight distance is the length of roadway needed for drivers on two-lane two-way highways to pass slower vehicles without meeting opposing traffic.

Passing Sight Distance Definitions

Vertical Curve Distance where an object (3.5 ft above roadway surface) can be seen from a point 3.5 ft above the roadway

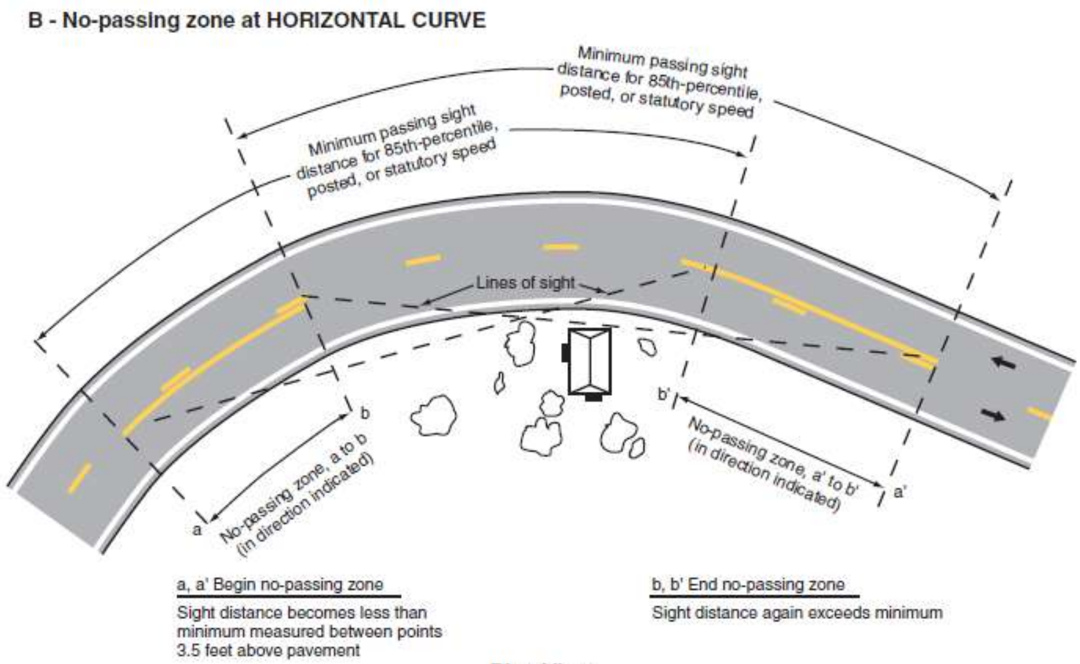
Horizontal Curve Distance measured (along center line or right-hand lane line for 3-lane roadway) between two points 3.5 ft above the roadway on a tangent line

Figure 3B-4. Method of Locating and Determining the Limits of No-Passing Zones at Curves



Profile View

Note: No-passing zones in opposite directions may or may not overlap, depending on alignment



Plan View

Note: No-passing zones in opposite directions may or may not overlap, depending on alignment

The following table shows design values for passing sight distance on two-lane highways. It has been shown that more sight distance is needed for passing maneuvers than for stopping sight distance which is continuously provided for along roadways.

Passing Sight Distance - Two-Lane Roadways

Design Speed (mph)	Assumed Speed (mph)		Passing Sight Distance (ft)
	Passed Vehicle	Passing Vehicle	
20	8	20	400
30	18	30	500
40	28	40	600
50	38	50	800
60	48	60	1000
70	58	70	1200
80	68	80	1400

Source: AASHTO "Green Book" Table 3-4

Potential passing conflicts are ultimately determined by driver responses to:

- View of roadway ahead
- Passing and no-passing zone markings

Horizontal alignment is also crucial to determine the location, extent, and percentage of passing distances. More sight distance is required for passing maneuvers than for stopping sight distance which is continuously provided along roadways.

Minimum values for passing sight distances are based on Manual on Uniform Traffic Control Devices (MUTCD) warrants for no-passing zones on two-lane highways. These values are suitable for single or isolated passes only.

<u>Driver Behavior Assumptions</u>

- Passing and opposing vehicle speeds are equal to the roadway design speed
- Speed differential between passing and passed vehicle is 12 mph
- Passing vehicle has adequate acceleration capability to reach speed differential (40% of way through passing maneuver)
- Vehicle lengths are 19 feet

- Passing driver's perception-reaction time to abort passing maneuver is 1 second
- Deceleration rate of 11.2 ft/s^2 for passing vehicle when passing maneuver is aborted
- Space headway between passing and passed vehicles is 1 second
- Minimum clearance between passing and opposed vehicles upon return to normal lane is 1 second

Design passing sight values should also be based on a single passenger vehicle passing another single passenger vehicle.

Passing sight distances should be should be as long and frequent as possible, and equal or greater than the minimum values, depending on:

- **topography**
- **design speed**
- **cost**
- **intersection spacing**

While passing sections are used on most highways and selected streets, others can usually be provided at little or no additional cost.

Comparison of Sight Distance Design Values

Design Speed (mph)	Passing Sight Distance (ft)	Stopping Sight Distance (ft)
20	400	115
30	500	200
40	600	305
50	800	425
60	1000	570
70	1200	730
80	1400	910

Source: AASHTO "Green Book" Figure 3-1

INTERSECTION SIGHT DISTANCE

The potential for vehicular conflicts at intersections can be greatly reduced with proper sight distances and traffic control. Intersection efficiency depends on driver behavior – judgment, capability, and response. Approaching drivers need an unobstructed view of the intersection and approaching roadways to safely maneuver through the facility.

Intersection sight distance is the length of roadway along the intersecting road that the approaching vehicle should have to perceive and react to potential conflicts. Both roadway horizontal and vertical geometry can have a great effect on ISD.

Sight distance is needed to allow stopped vehicles a sufficient view of the intersecting roadway in order to enter or cross it. Intersection sight distances that exceed stopping sight distances along major roads are considered sufficient to anticipate and avoid conflicts. Intersection sight distance determination is based on many of the same principles as stopping sight distance.

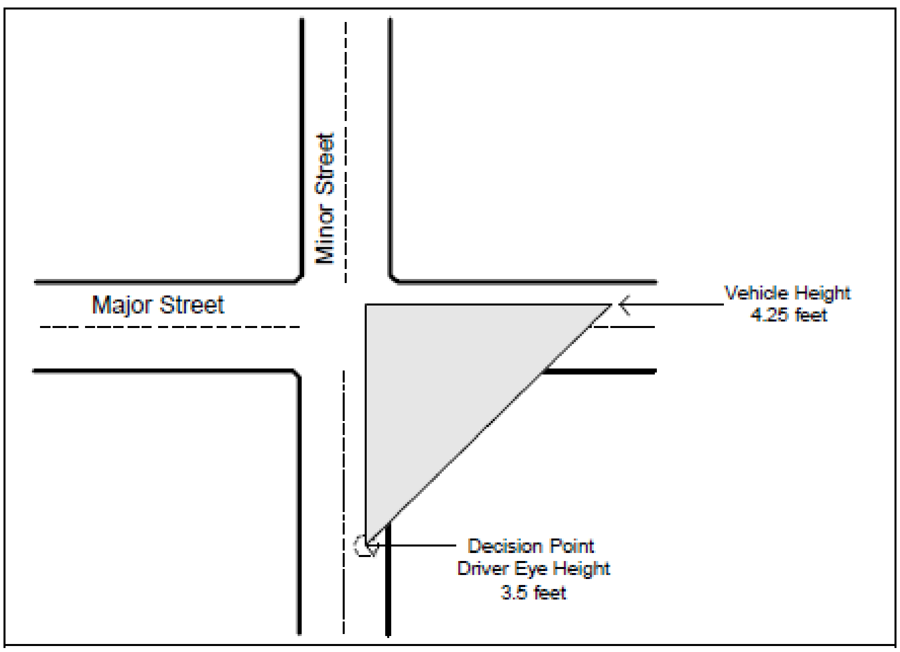


Figure 4.3. Heights Pertaining to Sight Triangles

Source: CTRE – Iowa State University

Clear sight triangles are areas along intersection approach legs that should be without any obstructions that could obscure any potential conflicts from the driver's view. For sight obstruction determination, the driver's eye is assumed to be **3.50 feet** above the road surface, and the visible object is **3.50 feet** above the intersecting road's surface. The dimensions are based on driver behavior, roadway design speeds, and type of traffic control. Object height is based on vehicle height of **4.35 feet** (15th percentile of current passenger vehicle height minus an allowance of 10 inches).

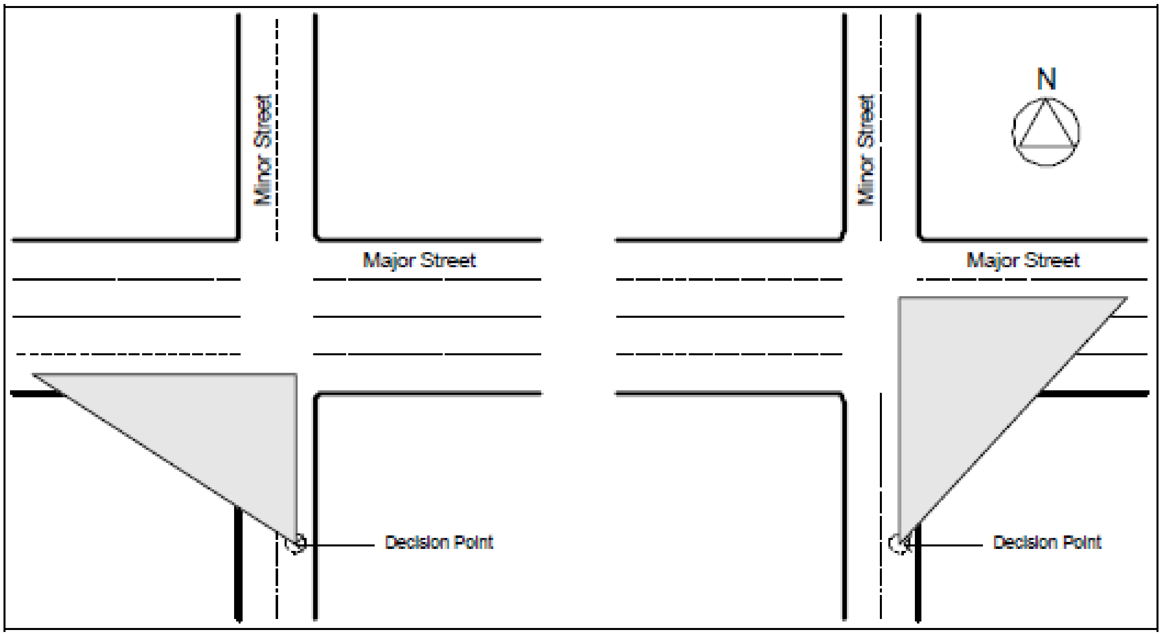


Figure 4.1. Approach Sight Triangles

Source: CTRE – Iowa State University

Approach sight triangles are triangular areas free of obstructions that could block approaching a motorist's view of potential conflicts. Lengths of the area legs should permit drivers to observe any potential conflicts and slow, stop, or avoid other vehicles within the intersection. These types of sight triangles are not needed for intersections controlled by stop signs or traffic signals.

Departure sight triangles provide adequate distance for stopped drivers on minor roads to depart the intersection and enter/cross the major road. These sight triangles are needed for quadrants of each intersection approach controlled by stop or yield conditions.

$$ISD = 1.47V_{major} + t_g$$

ISD = intersection sight distance (along major road) (feet)

V_{major} = design speed of major road (mph)

t_g = time gap for minor road vehicle to enter major road (seconds)

AASHTO's method for determining intersection sight distance is fairly complicated (speeds, traffic control, roadway cross-sections, obstruction location, vehicle types, maneuvers). Obstructions include building setbacks, trees, fences, etc. Railroad grade crossing sight distances to adjacent roadway intersections should also be addressed for intersection design and sight distance.

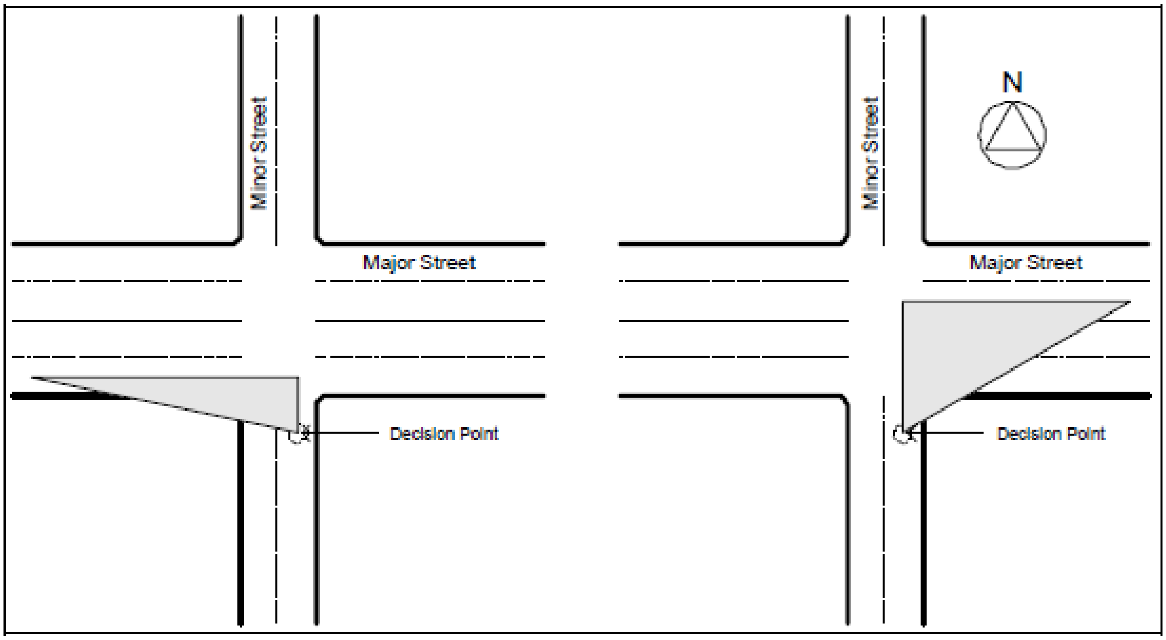


Figure 4.2. Departure Sight Triangles

Source: CTRE – Iowa State University

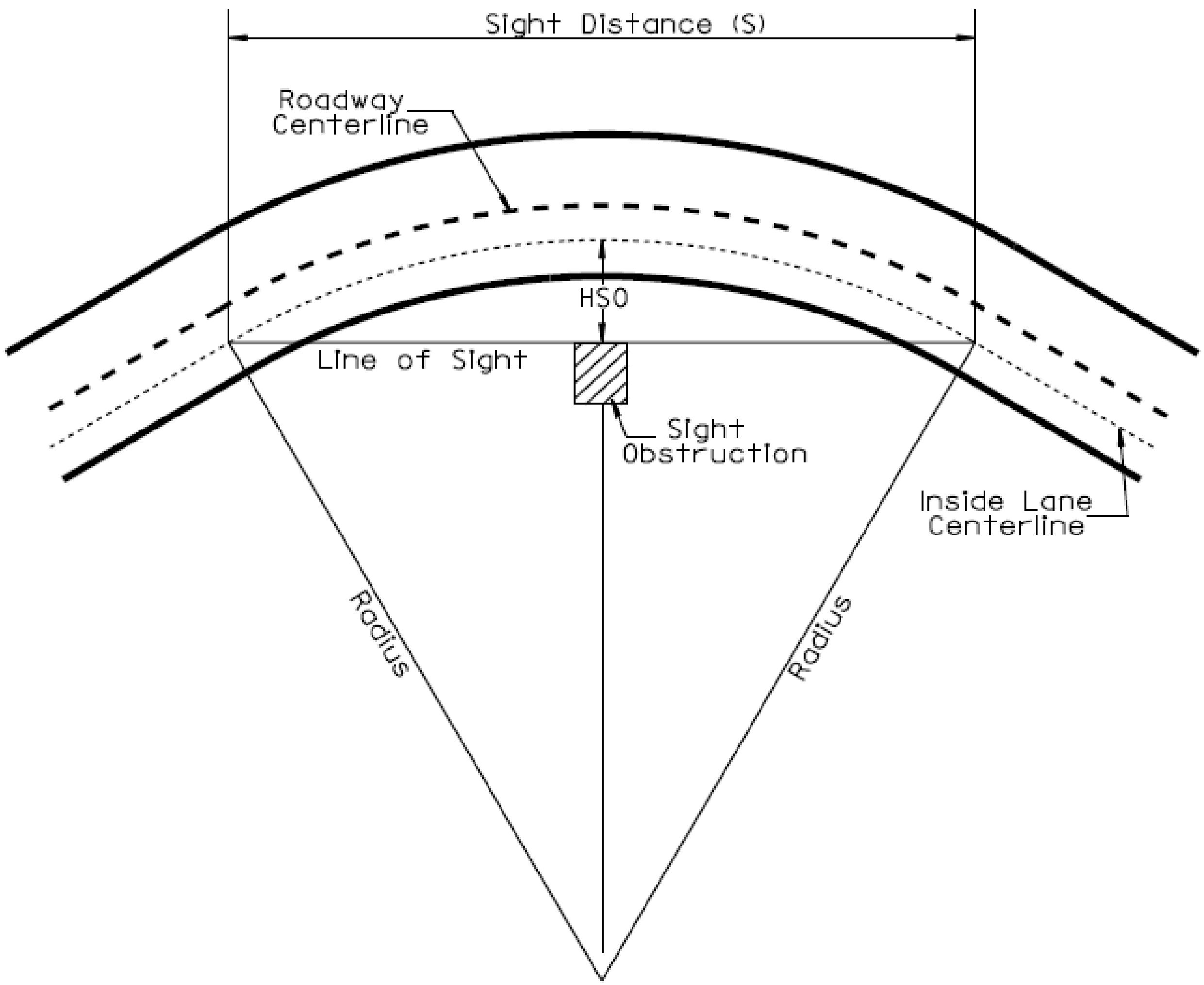
Methods for determining intersection sight distance vary according to the different types of traffic control:

- **Case A:** Intersections with no control
- **Case B:** Intersections with stop control on the minor road
  - **Case B1:** Left turn from the minor road
  - **Case B2:** Right turn from the minor road
  - **Case B3:** Crossing maneuver from the minor road
- **Case C:** Intersections with yield control on the minor road
  - **Case C1:** Crossing maneuver from the minor road
  - **Case C2:** Left or right turn from the minor road

- **Case D:** Intersections with traffic signal control
- **Case E:** Intersections with all-way stop control
- **Case F:** Left turns from the major road

Sight distance across the inside of curves is a crucial design control for horizontal alignments. Due to various concerns (alignments, cross-sections, obstructions, etc.), specific study is needed for each curve and adjustments made to provide sufficient sight distance.

For **horizontal alignments**, the sight line is a chord of the curve as shown below. The stopping sight distance is along the centerline of the curve's inside lane.



AASHTO Equation 3-36 is suitable for circular curve lengths greater than the sight distance for the design speed.

$$HSO = R\left[1 - cos\left(\frac{28.65S}{R}\right)\right]$$

HSO = horizontal sight line offset (feet)

R = radius of curve (feet)

S = stopping sight distance (feet)

Eye Height:	3.50 feet	
Object Height:	2.00 feet	Stopping sight distance
	2.75 feet	Midpoint of sight line where cut slope obstructs sight

The following alternatives may be considered where adequate stopping sight distance is not available:

- Increase offset to sight obstructions
- Increase curve radii
- Reduce design speed

Minimum passing sight distance values (two-lane road) are approximately twice those for the minimum stopping sight distance. Due to differences in sight line and stopping sight distance, design for passing sight distance should be limited to flat curves and tangents.

GENERAL CONTROLS

- **Any roadway alignment should be directional as possible.**

The horizontal alignment should be consistent with the topography and minimize any adverse impacts. Alignments consisting of short curves should be avoided since this may lead to erratic driving. Flowing centerlines that conform to the site's natural contours is generally preferable.

- **Avoid minimum radius values whenever possible.**

The central angle of all curves should be as small as practical to maximize roadway directionality. Typically, flat curves should be used with minimum radii for critical conditions.

- **Roadway alignment consistency is desirable.**

Sudden changes in the alignment should be avoided. For example, a series of successively sharp curves should be used to introduce a sharper curve.

- **Horizontal curves should be long enough for aesthetic purposes.**

These should avoid the appearance of a kink for small deflection angles. For a central angle of 5 degrees, curves should be a minimum of 500 feet long (with a minimum increase of 100 ft for each degree decrease in the central angle).

- **Avoid sharp curves on lengthy high embankments.**

The absence of other features (vegetation, cut slopes, etc.) makes it difficult for the driver to perceive and react to the extent of curvature.

- **Avoid changing median widths on tangent alignments.**

This will prevent distorted appearances.

- **Exercise caution when using compound circular curves.**

Compound curvature flexibility may tempt designers to use them without restraint. These curves should be avoided where curves are sharp.

- **Avoid sudden reversals in alignment.**

These changes make it difficult for safe operation (lane changes, etc.). Distances between reverse curves should be equal to the sum of the superelevation and tangent runout lengths, or an equivalent length for spiral curves.

- **Avoid "broken-back" or "flat-back" curve arrangements, where possible.**

These alignments containing a short tangent between two curves in the same direction usually violate operator expectations. Motorists generally do not expect successive curves in the same direction. Spiral transitions or compound curves are preferable for such situations.

- **Coordinate the horizontal alignment with the roadway profile.**

COORDINATION OF HORIZONTAL AND VERTICAL ALIGNMENTS

Geometric roadway design influences safety performance. Historical crash data has shown that roadway factors are the **second** most contributing factor to roadway accidents. Crashes are more likely to occur at locations with sudden changes in road character (i.e. sharp curves at the end of long tangent sections).

Design consistency compares adjacent road segments and identifies locations with changes that might violate driver expectations. This type of analysis can be used to show operating speed decreases at curves.

Horizontal and vertical geometrics are the most critical roadway design elements. These alignments should be designed concurrently to enhance

vehicle operation,

uniform speed,

and *aesthetics without additional costs.*

Examples include: checking for additional sight distance prior to major vertical alignment changes; or revising design elements to eliminate potential drainage problems.

Horizontal and vertical alignment geometric designs complement each other while poor designs can reduce the quality of both. It can be extremely difficult and costly to fix any vertical and/or horizontal deficiencies once a roadway is built. Any initial savings can be offset by economic losses due to crashes and delays.

Physical factors that help define roadway alignments include:

- Roadway traffic
- Topography
- Subsurface conditions
- Cultural development
- Roadway termini

Although design speed helps to determine the roadway's location, it assumes a greater role as the design of the horizontal and vertical alignments progress. Design speed aids in balancing all of the design elements by limiting many design values (curves, sight distance) and influencing others (width, clearance, maximum gradient).

GENERAL PROCEDURE

Coordinating horizontal and vertical alignments should begin with any roadway preliminary design. Any adjustments or corrections can be readily made at this phase.

Working drawings can be used for studying long, continuous plan and profile views to visualize the proposed three-dimensional roadway. Computer-aided drafting and design (CADD) systems are typically used to create optimal 3-D designs.

After development of a preliminary design, adjustments can be made for better coordination between the alignments. Using the design speed, the following factors should be checked:

Controlling curvature
Gradients
Sight distance
Superelevation runoff lengths

Also, the design controls for vertical and horizontal alignments should be considered, as well as all aspects of terrain, traffic, and appearance. All adjustments should be made before the costly and time-consuming preparation of construction plans.

For local roads, the alignment is impacted by existing or future development – with intersections and driveways being dominant controls. Designs should contain long, flowing alignments instead of a connected series of block-by-block sections.

AASHTO Design Guidelines for Horizontal and Vertical Alignments

- Vertical and horizontal elements should be balanced to optimize safety, capacity, operation, and aesthetics within the location's topography.

- Both horizontal and vertical alignment elements should be integrated to provide a pleasing facility for roadway traffic.

- Sharp horizontal curves near the top of a crest vertical curve or near the low point of a sag vertical curve should be avoided. Using higher design values (well above the minimum) for design speed can produce suitable designs and meet driver's expectations.

- Horizontal and vertical curves need to be as flat as possible for intersections with sight distance concerns.

- For divided roadways, it may be suitable to vary median widths for divided roadways. Independent horizontal/vertical alignments should be used for individual one-way roads.

- Horizontal and vertical alignments should be designed to minimize impact in residential areas. Typical applications include:

  depressed facilities (decreases facility visibility and noise)

  horizontal adjustments (increases buffer zones between traffic and neighborhoods).

- Geometric design elements should be used to enhance environmental features (parks, rivers, terrain, etc.). Roadways should enhance outstanding views or features instead of avoiding them where possible.

Exception: Long tangent sections for sufficient passing sight distance may be appropriate for two-lane roads needing passing sections at frequent intervals.

SUMMARY

Along with the roadway cross section (lanes and shoulders, curbs, medians, roadside slopes and ditches, sidewalks) and vertical alignment (grades and vertical curves), the **horizontal alignment** (tangents and curves) helps provide a three-dimensional roadway model. Its ultimate goal is to provide a safe, smooth-flowing facility that is crash-free. Roadway horizontal alignments are directly related to their operational quality and safety.

In today's environment, designers must do more than apply design standards and criteria to 'solve' a problem. They must understand how various roadway elements contribute to safety and facility operation, including the horizontal alignment.

This course summarizes the geometric design of horizontal alignments for modern roads and highways. This document is intended to serve as guidance and not as an absolute standard or rule. For further information, please refer to **AASHTO's A Policy on Geometric Design of Highways and Streets (Green Book)**. It is considered to be the primary guidance for U.S. roadway design. **Section 3.3 – Horizontal Alignment** was used exclusively to present fundamental horizontal roadway geometric design principles.

By completing this course, you should be familiar with the general design of horizontal roadway alignments. The objective of this course was to give engineers and designers an in-depth look at the principles to be considered when selecting and designing roads.

This course focused on the following:

- Sight Distance
  - *Stopping*
  - *Decision*
  - *Passing*
  - *Intersection*
- Design Considerations
  - *Cross slopes*
  - *Superelevation*
  - *Radii*
  - *Grades*

- Horizontal Curves
  Compound
  Spiral

- Coordination of Horizontal & Vertical Curves

The fundamental objective of good geometric design will remain as it has always been – **to produce a roadway that is safe, efficient, reasonably economic and sensitive to conflicting concerns.**

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Note: *This text is the source for all equations and tables contained within this course, unless noted otherwise.*

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