
Geometric Design Elements

A. Design Flexibility and Performance-Based Approach

The 2018 AASHTO Green Book introduced a new design process that takes a performance-based approach to design. Performance-based design is problem-driven, and helps designers prioritize certain goals in order to make decisions and tradeoffs to best meet the needs of all transportation modes. In nearly all road construction projects, there are constraints requiring tradeoffs that do not allow the use of all the preferred design criteria presented in [Section 5C-1](#). Designers should consult the design criteria set forth in this section, but it may be impractical to use them all because of existing constraints in the corridor and the need to fit the roadway into the community context and meet the needs of all transportation modes. For example, if analysis of the crash history of the existing road identifies one or more crash patterns that are potentially correctable by a specific design improvement, the designer may place greater priority on incorporating design improvements to reduce those crashes, at the expense of maintaining or improving the traffic operational level of service (see below).

[Section 5M-1](#) provides more background on applying flexibility in street design to meet project goals.

B. Level of Service

Level of service (LOS) measures motor vehicle delay for a roadway facility and is one measure to assess roadway performance. It should be considered along with other factors to reflect the goals of the project. LOS is based upon motor vehicle traffic conditions and is related to speed, travel time, freedom to maneuver, traffic interruptions, and comfort and convenience. The LOS scale ranges from A (least congested) to F (most congested). Refer to the Highway Capacity Manual for a more thorough discussion of the LOS concept.

Based upon a traffic capacity analysis, the number of lanes, turn lanes, and intersection controls should be selected to provide a design with the desired LOS for the design year traffic. Designers should select a design year and target LOS based on the context of the project location, the goals of the project, and modes present. The current Highway Capacity Manual, the current AASHTO Green Book, and other references should be used for traffic projections and to determine the number of lanes and intersection configurations for the target LOS. Designers should refer to Table 5C-1.01 and 5C-1.02 for target design LOS and [Section 5M-1, C, 3](#) for guidance on appropriately sizing roadways in urban, suburban, and rural town contexts.

The LOS for the roadway overall is determined based on Average Daily Traffic (ADT), while the LOS at signalized intersections is determined based on the peak hourly volume (PHV).

As a planning tool, refer to the generalized service volume tables in FHWA's [Simplified Highway Capacity Calculation Method for the Highway Performance Monitoring System](#).

The 2010 Highway Capacity Manual, issued in 2013, indicates there is no reduction in lane capacity until the lane width is less than 10 feet. For lanes less than 10 feet wide, the adjustment factor is 0.96.

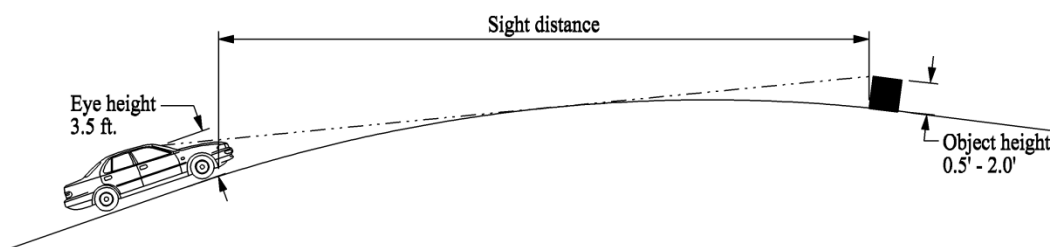
C. Sight Distance

The following information is taken from the 2004 AASHTO Green Book. The Project Engineer should check the current edition of the AASHTO Green Book when specific information is needed to verify values provided.

1. **Stopping Sight Distances:** The minimum stopping sight distance is the distance required by the driver of a vehicle traveling at the design speed to bring the vehicle to a stop after an object on the road becomes visible. This distance directly affects the length and rate of curvature for vertical curves.

The method for measuring stopping sight distance on vertical curves assumes a height for the driver's eye and a height for an object in the road. For a crest vertical curve, the sight distance is the distance at which an object in the road appears to the driver over the crest of the curve.

Figure 5C-2.01: Vertical Sight Distance Determination



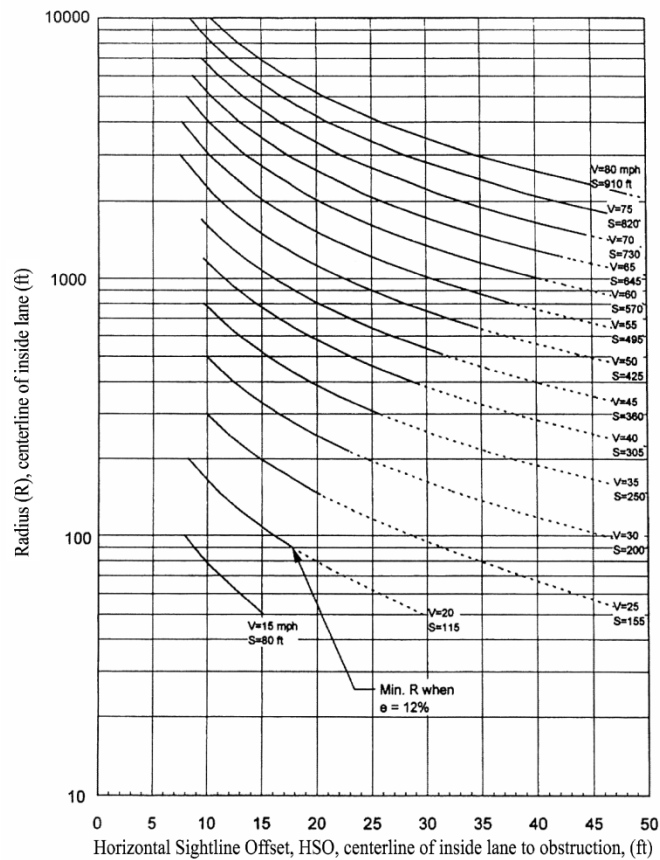
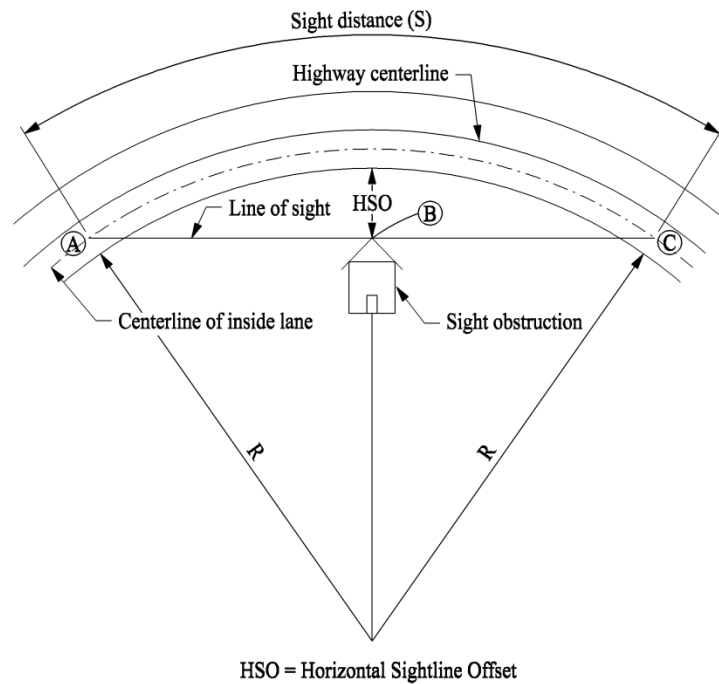
Stopping sight distance is calculated based upon an assumed height of the driver's eye and an assumed height of an object in the roadway. For all sight distance criteria, the height of the driver's eye is assumed to be 3.5 feet above the surface of the road, as recommended by AASHTO. [Section 5C-1](#), [Tables 5C-1.01](#) and [5C-1.02](#) assume two different values for the height of the object in the roadway. The "Acceptable" values in [Table 5C-1.02](#) use a 2 foot object height according to the current edition of the AASHTO Green Book. The "Preferred" values in [Table 5C-1.01](#) assume an object height of only 6 inches. This lower object height was the design value used in previous versions of the AASHTO Green Book. The results of assuming a smaller object height for the preferred values in [Table 5C-1.01](#) are higher required K values and longer vertical curves.

2. **Sight Distance on Horizontal Curves:** The horizontal alignment must provide at least the minimum stopping distance for the design speed at all points. This includes visibility around curves and roadside encroachments.

Where there are sight obstructions such as walls, cut slopes, buildings, fences, bridge structures, or other longitudinal barriers on the inside of curves, an adjustment in the minimum radius of the curve may be necessary. In no case should sight distance be less than the stopping sight distance specified in [Section 5C-1](#), [Tables 5C-1.01](#) and [5C-1.02](#). The sight distance design procedure should assume a 6 foot fence (as measured from finished grade) exists along all property lines except in the sight distance triangles required at all intersections.

Available sight distance around a horizontal curve can be determined graphically using the method shown in Figures 5C-2.02 and 5C-2.03 below. From the center of the inside lane (Point A), a line is projected through the point on the obstruction that is nearest to the curve (Point B). The line is then extended until it intersects the centerline of the inside lane (Point C).

Figure 5C-2.02 and Figure 5C-2.03: Sight Distances for Horizontal Curves



Source: Adapted from AASHTO Green Book, 2004 Edition, Exhibits 3-53 and 3-54

3. **Passing Sight Distance:** Passing sight distance is the minimum sight distance that must be available to enable the driver of one vehicle to pass another safely and comfortably without interfering with oncoming traffic traveling at the design speed. Two lane roads should provide adequate passing zones at regular intervals. Minimum passing sight distances are shown in [Section 5C-1](#), [Tables 5C-1.01](#) and [5C-1.02](#).

Passing sight distance is measured between an eye height of 3.5 feet and an object height of 3.5 feet. On straight sections of roadway, passing sight distance is determined primarily by the vertical curvature of the roadway. On horizontal curves, obstructions adjacent to the roadway on the inside of the curve can limit sight distance. This is most common in a cut section where the adjacent terrain projects above the surface of the roadway. Passing sight distance should be verified using the methods described in the current edition of the AASHTO Green Book.

4. **Intersection Sight Distance:** In addition to the stopping sight distance provided continuously in the direction of travel on all roadways, adequate sight distance at intersections must be provided to allow drivers to perceive the presence of potentially conflicting vehicles. Sight distance is also required at intersections to allow drivers of stopped vehicles to decide when to enter or cross the intersecting roadway. If the available sight distance for an entering or crossing vehicle is at least equal to the appropriate stopping sight distance for the major road, then drivers have sufficient sight distance to anticipate and avoid collisions. However, in some cases, this may require a major road vehicle to slow or stop to accommodate the maneuver by a minor road vehicle. To enhance traffic operations, intersection sight distances that exceed stopping sight distances are desirable along the major road.

Each intersection has the potential for several different types of vehicular conflicts. The possibility of these conflicts actually occurring can be greatly reduced by providing proper sight distance and appropriate traffic controls. Each quadrant of an intersection should contain a triangular area free of obstructions that might block an approaching driver's view of potentially conflicting vehicles. This clear area is known as the sight triangle.

- a. **Sight Triangles:** Proper sight distance at intersections is determined through the establishment and enforcement of sight triangles. The required dimensions of the legs of the triangle depend on the design speed of the roadways and the type of traffic control provided at the intersection. Two types of clear sight triangles are considered in intersection design: approach sight triangles and departure sight triangles.
 - 1) **Approach Sight Triangles:** Approach sight triangles allow the drivers at uncontrolled or yield controlled intersections to see a potentially conflicting vehicle in sufficient time to slow or stop before colliding within the intersection. Although desirable at all intersections, approach sight triangles are not needed for intersections approaches controlled by stop signs or traffic signals.
 - 2) **Departure Sight Triangles:** A second type of clear sight triangle provides sight distance sufficient for a stopped driver on a minor-road approach to depart from the intersection and enter or cross the major road. Departure sight triangles should be provided in each quadrant of each intersection approach controlled by a stop sign.

At signalized intersections, the first vehicle stopped on one approach should be visible to the driver of the first vehicle stopped on each of the other approaches. Left turning vehicles should have sufficient sight distance to select gaps in oncoming traffic.

The recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection because different types of controls impose different legal constraints on drivers and, therefore, result in different driver behavior. The AASHTO Green Book contains

the required procedures, equations, and tables for determining the required sight distance under various intersection and traffic control configurations.

- b. Identification of Sight Obstructions within Sight Triangles:** Within a sight triangle, any object at a height above the elevation of the adjacent roadways that would obstruct the driver's view should be removed or lowered if practical. Such objects may include buildings, parked vehicles, highway structures, roadside hardware, hedges, trees, bushes, unmowed grass, tall crops, walls, fences, and the terrain itself. Particular attention should be given to the evaluation of clear sight triangles at intersection ramp/crossroad intersections where features such as bridge railings, piers, and abutments are potential sight obstructions.

The determination of whether an object constitutes a sight obstruction should consider both the horizontal and vertical alignment of both intersecting roadways, as well as the height and position of the object. In making this determination, it should be assumed that the driver's eye is 3.5 feet above the roadway surface and that the approaching vehicle to be seen is 3.5 feet above the surface of the intersecting road.

D. Horizontal Alignment

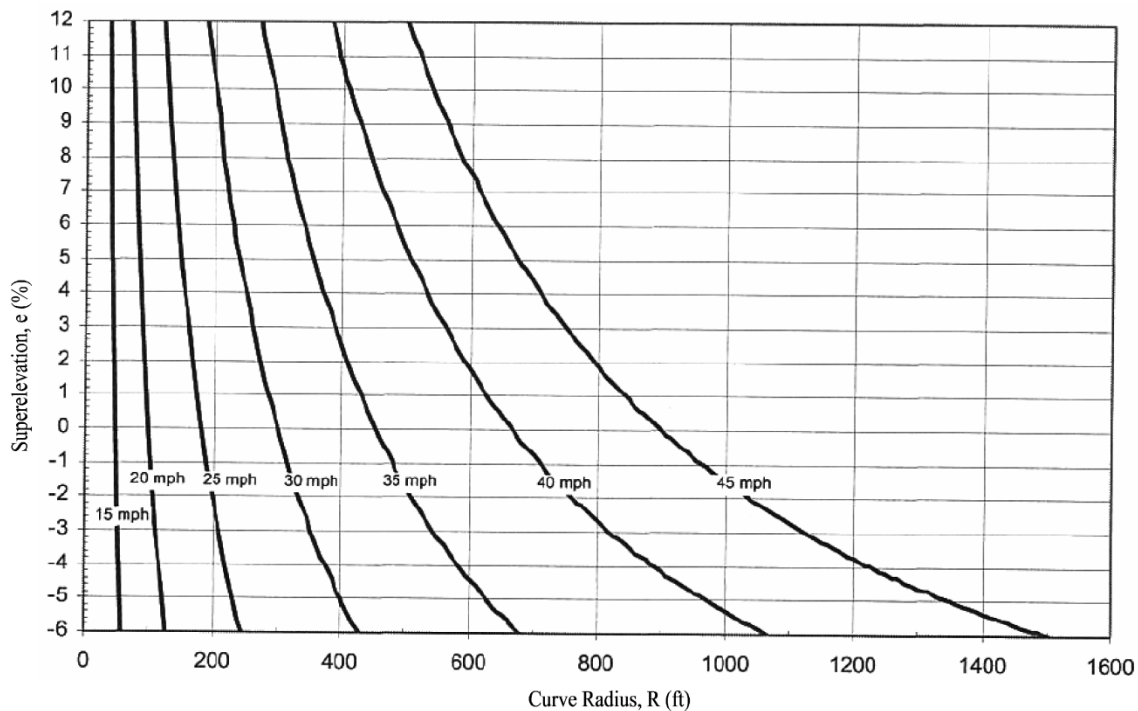
- 1. Roadway Curvature and Superelevation:** On urban streets where operating speed is relatively low and variable, the use of superelevation for horizontal curves can be minimized. Although superelevation is advantageous for traffic operation, in urban areas the combination of wide pavements, the need to meet the grade of adjacent properties, the desire to maintain low speed operation, the need to maintain pavement profiles for drainage, and the frequency of cross streets and driveways and other urban features often combine to make the use of superelevation impractical or undesirable. Generally, the absence of superelevation on low speed urban streets is not detrimental to the motorist and superelevation is not typically provided on urban streets with a design speed of 45 mph or less.

The preferred radii shown in [Section 5C-1, Table 5C-1.01](#) assume that a normal crown is maintained around a horizontal curve. With a standard 2% pavement cross-slope, this effectively results in a negative 2% superelevation for the outside lane. For roadways with a cross-slope other than 2%, including four lane and wider sections that utilize a steeper cross-slope for the outside lanes, the required curve radius should be determined from the guidance provided in the current AASHTO Green Book or from Figure 5C-2.04 below.

While superelevation on low speed urban roadways is not desirable, it may be necessary in situations where site conditions require a horizontal curve that cannot sustain traffic with the negative superelevation that results from maintaining the normal crown. For these situations, superelevation equal to the normal cross-slope may be provided for the outside lane. [Section 5C-1, Table 5C-1.02](#) assumes the adverse crown in the outside lane of a curve is removed. For a roadway with a normal 2% cross-slope, this results in a superelevation of 2% across the width of the pavement. For roadways with cross-slopes other than 2%, the required radius and the resulting superelevation should be determined from the guidance provided in current AASHTO Green Book or from Figure 5C-2.04 below. The maximum superelevation for low speed urban roadways should not exceed the normal cross-slope or a maximum of 3%.

For roadways with design speeds of 50 mph or greater, superelevation of the roadway is acceptable and expected by motorists. The radii provided in [Section 5C-1, Tables 5C-1.01 and 5C-1.02](#) are based upon superelevation rates of 4% and 6% respectively. The maximum superelevation rate in urban areas should not exceed 6%.

Figure 5C-2.04: Superelevation, Radius, and Design Speed for Low Speed (<50mph)
Urban Street Design



Source: AASHTO Green Book, 2004 Edition, Exhibit 3-17

2. **Intersection Alignment:** The centerline of a street approaching another street from the opposite side should not be offset. If the offset cannot be avoided, the offset should be 150 feet or greater for local streets. The centerline of a local street approaching an arterial or collector street from opposite side should not be offset unless such offset is 300 feet or greater.

3. Adding, Dropping, or Redirecting Lanes:

- a. **Dropping or Redirecting Through Lanes:** When dropping a lane, the minimum taper ratio to be used should be determined by the following formula, or from Table 5C-2.01:

$L = WS$ for velocities of 45 mph or more

$L = \frac{WS^2}{60}$ for velocities of 40 mph or less.

L = Minimum length of taper.

S = Numerical value of posted speed limit or 85th percentile speed, whichever is higher.

W = Width of pavement to be dropped or redirection offset.

Preferably, taper ratios should be evenly divisible by five. Calculations that result in odd ratios should be rounded to an even increment of five. The table below utilizes the formulas to determine the appropriate taper ratio for dropping a 12 foot wide lane. The ratio remains constant for a given design speed while the length varies with the pavement width.

The procedure for determining minimum taper ratios for redirecting through lanes is the same as for lane drops, except for design speeds over 45 mph the use of reverse curves rather than tapers is recommended.

Table 5C-2.01: Length and Taper Ratio for Dropping 12 Foot Lane

Design Speed (mph)	25	30	35	40	45	50	55	60
Taper Ratio	10:1	15:1	20:1	25:1	45:1	50:1	55:1	60:1
Length (feet)	120	180	240	300	540	600	660	720

- b. Adding Through or Turn Lanes:** For design speeds of 45 mph or greater, a 15:1 lane taper should be used when adding a left or right turn lane. For design speeds less than 45 mph, a 10:1 taper may be used.

For design speeds less than 45 mph, shorter tapers that are squared off or taper at 1:1 may provide better “targets” for approaching drivers and give more positive identification to an added through lane or turn lane. For turn lanes, the total length of taper and deceleration length should be the same as if a standard taper was used. This results in a longer length of full width pavement for the turn lane. This design provides increased storage that may reduce the likelihood turning vehicles will back up into the through lane during peak traffic periods. The use of short taper sections must be approved by the Engineer.

Figure 5C-2.05: Adding or Dropping Lanes

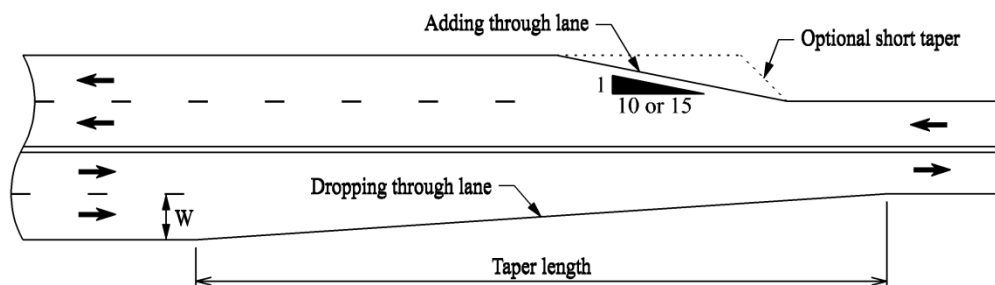
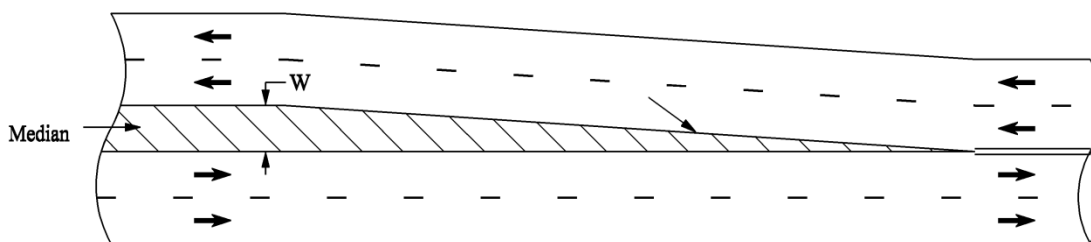


Figure 5C-2.06: Redirecting Through Lanes



E. Vertical Alignment

1. **Minimum Grades:** Flat and level grades on uncurbed pavements are preferred when the pavement is adequately crowned to drain the surface laterally. However, with curbed pavements, longitudinal grades must be provided to facilitate surface drainage. A typical minimum grade is 0.5%, but a grade of 0.4% may be used in isolated areas where the pavement is accurately crowned and supported on firm subgrade. The minimum allowance grade for bubbles and cul-de-sacs is 1%. Particular attention should be given to the design of stormwater inlets and their spacing to keep the spread of water on the traveled way within tolerable limits. Roadside channels and median swales frequently require grades steeper than the roadway profile for adequate drainage.
2. **Maximum Grades:** Grades for urban streets should be as level as practical, consistent with the surrounding terrain. The maximum design grades specified in [Section 5C-1](#), [Table 5C-1.02](#) should be used infrequently; in most cases grades should be less than the maximum design grade.

Where sidewalks are located adjacent to a roadway, a maximum roadway grade of 5% is desirable. ADA requirements allow sidewalks adjacent to a roadway to match the running grade of the roadway, regardless of the resulting grade. However, sidewalk accessibility is greatly enhanced, especially over long distances, when grades are limited to 5% or less. It is recognized that meeting limitations will not be possible or practical in many situations; however, an attempt should be made to limit roadway grades to this level, especially in areas with high levels of anticipated pedestrian usage.

3. **Maximum Grade Changes:** Except at intersections, the use of grade breaks, in lieu of vertical curves, is not encouraged. However, if a grade break is necessary and the algebraic difference in grade does not exceed 1%, the grade break will be considered by the Engineer.
4. **Vertical Curves:** Vertical curves should be simple in application and should result in a design that is safe, comfortable in operation, pleasing in appearance, and adequate for drainage.

The major control for safe operation on crest vertical curves is the provision of ample sight distances for the design speed. Minimum stopping sight distance should be provided in all cases. Wherever economically and physically feasible, more liberal stopping sight distances should be used. Furthermore additional sight distance should be provided at decision points.

- a. **Crest Vertical Curves:** Minimum lengths of crest vertical curves as determined by sight distance requirements are generally satisfactory from the standpoint of safety, comfort, and appearance. Figure 5C-2.06 shows the required length of crest vertical curve to provide stopping sight distance based upon design speed and change in grade.
- b. **Sag Vertical Curves:** Headlight sight distance is generally used as the criteria for determining the length of sag vertical curves. When a vehicle approaches a sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. A headlight height of 2 feet and a 1 degree upward divergence of the light beam from the longitudinal axis of the vehicle is commonly assumed. For safety purposes, the sag vertical curve should be long enough that the light beam distance is the same as the stopping sight distance. Figure 5C-2.07 specifies the required sag curve length to meet the sight distance assumptions made above.

For both sag and crest vertical curves with a low algebraic difference in grade, sight distance restrictions may not control the design of the curve. In these cases, rider comfort and curve appearance are the primary considerations for vertical curve design. Generally, vertical curves with a minimum length (in feet) equal to three times the design speed (in mph) are acceptable.

If a roadway has continuous lighting, the length of sag vertical curve (L) may be based on passenger comfort instead of headlight sight distance. Use the following equation for the curve length:

$$L = \frac{AV^2}{46.5} \quad \text{where } A = \text{algebraic difference in grades, \%}$$

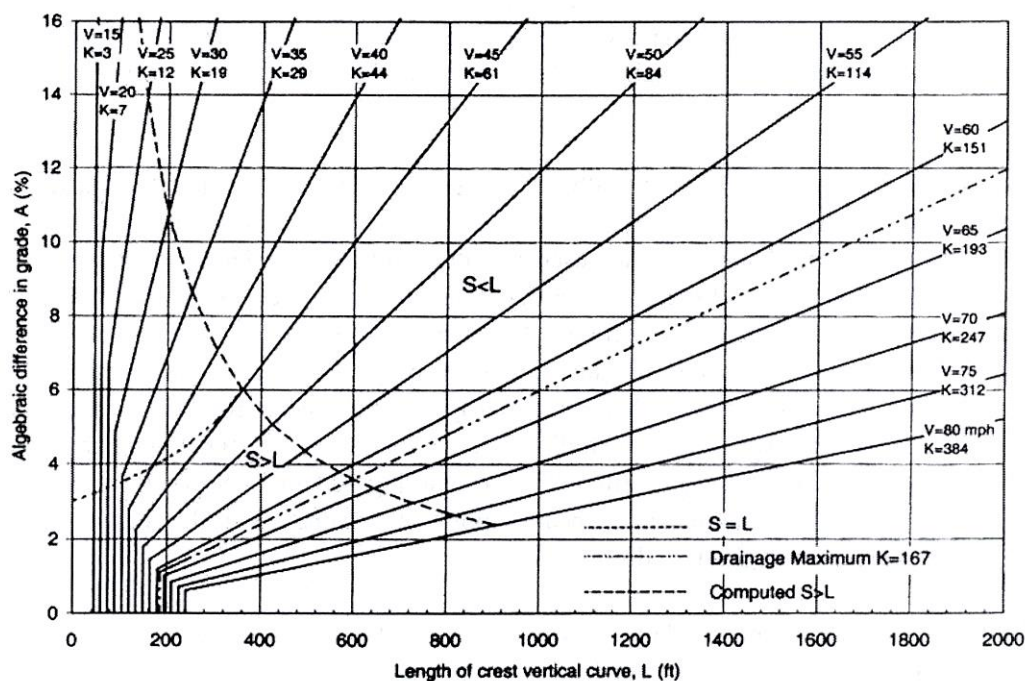
$$V = \text{design speed, mph}$$

(Equation 3-51 AASHTO Green Book, 2011)

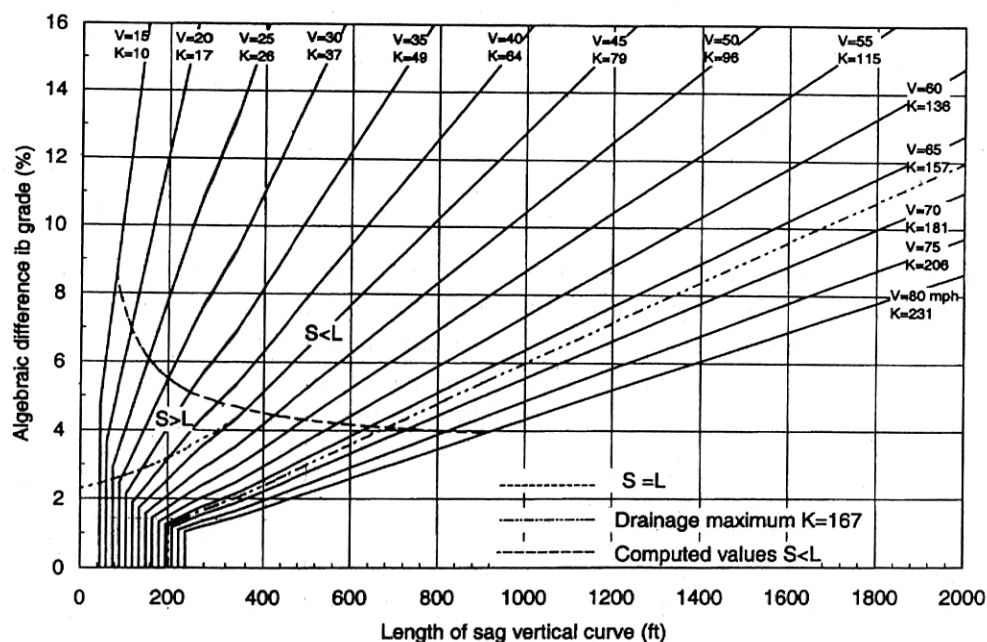
Drainage considerations also affect the design of vertical curves where curbs are utilized. Both crest and sag vertical curves that have a grade change from positive to negative (or vice versa) contain a level area at some point along the curve. Generally, as long as a grade of 0.30% is provided within 50 feet of the level area, no drainage problems develop. This criterion corresponds to a K value of 167 and is indicated by a dashed line in Figures 5C-2.06 and 5C-2.07 below. K values greater than 167 may be utilized, but additional consideration should be given to drainage in these situations.

$$K = \frac{L(\text{ft})}{(g_2 - g_1)} \quad \text{where } g_1 \text{ and } g_2 \text{ are in percent}$$

Figure 5C-2.07: Design Controls for Crest Vertical Curves for Stopping Sight Distance and Open Road Conditions



Source: AASHTO Green Book, Exhibit 3-71, 2004

Figure 5C-2.08: Design Controls for Sag Vertical Curves, Open Road Conditions

Source: AASHTO Green Book, Exhibit 3-78, 2004

5. **Intersection Grades:** The grade of the "through" street should take precedence at intersections. At intersections of roadways with the same classifications, the more important roadway should have this precedence. Side streets are to be warped to match through streets with as short a transition as possible, which provides a smooth ride. Consideration must be given to minimize sheet flow of stormwater across the intersection due to loss of crown on the side street.

Carrying the crown of the side street into the through street is not allowed. In most cases the pavement cross-slope at the warped intersection should not exceed the grade of the through street.

The maximum desirable grades of the through street at the intersection and the side street cross-slope should be 2% and should not exceed 3%. The maximum desirable approach grade of the side street should not exceed 4% for a distance of 100 feet from the curb of the through street.

Establishing intersection spot grades by matching "curb corners" of intersecting streets is not recommended since it may result in an undesirable travel path from the through street to the side street because of the resulting bump on the side street centerline. At sidewalk curb ramps in intersections, the street grades may need to be warped at the curb line to ensure the resulting cross-slope at the bottom of the ramp does not exceed 2%. A detail of the jointing layout with staking elevations should be shown on the plans.

ADA regulations set specific limits for crosswalk cross-slopes that directly impact street and intersection grades. ADA regulations limit the cross-slope to 2% (measured perpendicular to the direction of pedestrian travel) for crosswalks that cross a roadway with stop control (stop sign) at the intersection. For roadways without stop control (through movement or traffic signal) the cross-slope of the crosswalk is limited to 5%. Effectively, this requirement limits street grades to a maximum of 2% or 5% depending on intersection controls.

For steep roadways without stop control, construction of a flattened "table" may be necessary to reduce the street grade to 5% or less at the location of the crosswalk. Crosswalk tables at these

locations must utilize vertical curves, appropriate for the design speed, to avoid a sudden change in grade at the intersection that could cause vehicles to bottom out or lose control.

For steep roadways with stop control, construction of a flattened “table” may utilize grade breaks or shortened vertical curves to reduce the street grade to 2% or less at the location of the crosswalk. A check should be made to verify that vehicles will not bottom out when traveling over the crosswalk table.

F. Pavement Crowns

The following typical pavement crowns are straight line cross-slope and are desirable sections.

1. **Urban Roadways (Curb and Gutter):** For streets with three or fewer travel lanes, the pavement crown should be 2%.

For streets with four or more travel lanes, the pavement crown for all inside lanes, including left turn lanes, should be 2%. In order to reduce stormwater spread, the pavement crown for the outside lanes should be 3%.

For all streets, auxiliary right turn lanes will have varying pavement crowns depending on the desired drainage pathway.

2. **Rural Roadways:** For pavement crowns, a 2% cross-slope is normal with 4% shoulder slope. Where sidewalks are not present and shoulders are designated as the pedestrian access route, they must not exceed 2% cross slope per ADA requirements. [Iowa DOT Standard Road Plans](#) should be checked for Federal Aid, Farm to Market, and Secondary Roads.

G. Lane Width

The lane width of a roadway influences driver speed. Narrow lanes force drivers to operate their vehicles closer to each other laterally than they would normally desire, resulting in lower operating speeds.

[Section 5C-1](#), [Tables 5C-1.01](#) and [5C-1.02](#) indicate minimum lane widths based on the roadway classification and adjacent land use. Refer to [Section 5M-1](#) for additional discussion of lane width for multimodal streets.

Auxiliary lane and turn lane widths at intersections should be equal to the adjacent through lanes. The width for turn lanes is measured to the face of curb. Because motorists are slowing in anticipation of making a turning movement, drivers are comfortable operating their vehicle closer to an adjacent obstacle (curb); therefore, turn lanes do not require a curb offset.

H. Two-way Left-turn Lanes (TWLTL)

Two-way left-turn lanes work well where design speeds are relatively low (25 to 50 mph) and there are no heavy concentrations of left turning traffic. The width of TWLTLs should be limited to a maximum of 14 feet to discourage left-turning motorists from pulling out into the TWLTL and stopping perpendicular to the direction of traffic, while they wait for oncoming traffic to clear. [Section 5C-1](#), [Tables 5C-1.01](#) and [5C-1.02](#) indicate preferred and acceptable widths for TWLTLs.

I. Raised Median Width

A median is defined as the portion of a roadway separating opposing directions of the traveled way. The median width is expressed as the dimension between the edges of the traveled way and includes the left turn lanes, if any are present (refer to [Section 5C-1](#), [Figure 5C-1.01](#)). The principal functions of a median are to separate opposing traffic, allow space for speed changes and storage of left turning and U-turning vehicles, minimize headlight glare, and provide width for future lanes. For maximum efficiency, a median should be highly visible both night and day and contrast with the traveled way lanes.

At unsignalized intersections on rural divided highways, the median should generally be as wide as practical. However, in urban areas, narrower medians appear to operate better at unsignalized intersections. If right-of-way is restricted, a wide median may not be justified if provided at the expense of a narrowed border area. A reasonable border width is needed to adequately serve as a buffer between private development along the road and the traveled way. Narrowing the border area may create operational issues similar to those that the median is designed to avoid. In addition, wide medians at signalized intersections result in increased time for pedestrians, bicyclists, and vehicles to cross the median. This can lead to an increase in exposure time for pedestrians resulting in reduced safety and inefficient signal operation. Therefore, in urban areas, it is recommended that median width be only as wide as necessary to accommodate left turn lanes. Wider medians should only be used where needed to accommodate turning and crossing maneuvers by larger vehicles.

Medians and boulevards are not normally used on collector streets. However, when allowed, the median or boulevard should conform to the same design standards as set forth for arterial streets.

Median widths are also affected by sidewalk and crosswalk locations. Where a crosswalk cut through is present or proposed, the minimum width for a crossing island to provide an accessible refuge is 6 feet, measured from outside edge of the detectable warning surfaces, and the minimum width between detectable warning surfaces is 24 inches. The detectable warnings must be separated by a minimum 2 foot strip without detectable warnings. Where the median has no curb, the detectable warnings must be placed along the edge of the roadway. At locations where a raised median is stopped short of the crosswalk, the 6 foot raised median and associated detectable warnings are not required, and a standard 4 foot raised median section may be used. See Section 12A-5 for more guidance on designing safe pedestrian and bicycle crossings at medians.

J. Bridges

The bridge widths listed in [Section 5C-1](#), [Tables 5C-1.01](#) and [5C-1.02](#) represent the clear roadway width (width between barrier rail faces). The widths shown do not account for barrier rail widths, sidewalk, recreational trails, etc. In addition, designers should consider a sidewalk on one side in rural areas and should include sidewalks on both sides in rural town, suburban, and urban areas, even when sidewalks may not exist today. Sidewalks are difficult to retrofit on existing bridges and not including them now may preclude future sidewalk connections. For bridges with a shared use path, see [Section 12B-2, 1](#).

For existing bridges, a structural analysis should be conducted. The existing bridge should be able to accommodate legal loads. Bridge guardrail should be upgraded if necessary.

K. Clear Zone

The AASHTO Roadside Design Guide (RDG) defines the clear zone as “the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear runout area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry.”

The intent of the clear zone is to provide an errant vehicle that leaves the roadway with an unobstructed recovery area. This area, including medians on divided roadways, should be kept free of all unyielding objects, including utility and light poles, culverts, bridge piers, sign supports, and any other fixed objects that might severely damage an out of control vehicle. Any obstruction that cannot be placed outside of the clear zone should be shielded by traffic barriers or guardrails.

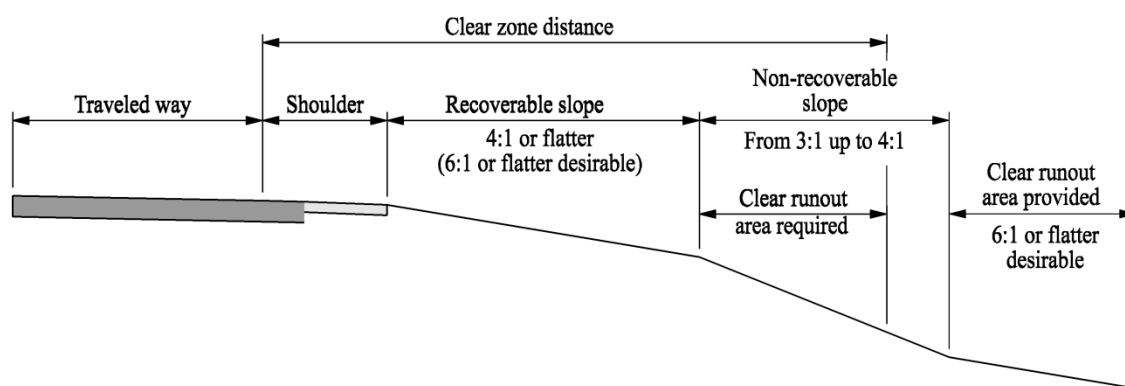
According to the AASHTO RDG, the width of this area varies based upon traffic volumes, design speed, and embankment slope.

Embankment slopes can be classified as recoverable, non-recoverable, or critical. Embankment slopes of 4:1 and flatter are considered recoverable. Drivers who encroach on recoverable slopes can generally stop their vehicles or slow them enough to return to the roadway safely.

A non-recoverable slope is defined as one that is passable, but from which most motorists will be unable to stop or to return to the roadway easily. Vehicles on such slopes are likely to reach the bottom before stopping. Embankments between 3:1 and 4:1 generally fall into this category. Since many vehicles will reach the toe of these slopes, the clear zone distance cannot logically end on a non-recoverable slope, and a clear runout area at the base of the slope is required. Fixed objects should not be present on a non-recoverable slope.

A critical slope is one on which a vehicle is likely to overturn. Slopes steeper than 3:1 generally fall into this category. If a slope steeper than 3:1 begins closer to the traveled way than the suggested clear zone, a barrier might be warranted if the slope cannot be flattened.

Figure 5C-2.09: Clear Zone Components



Source: Adapted from *Roadside Design Guide*, 2006

For horizontal curves, an adjustment factor may be applied to the clear zone width taken from [Section 5C-1](#), [Tables 5C-1.03](#) or [5C-1.04](#). This adjustment is only required at selected locations. Widening the clear zone should be considered along the outside of curves when crash history suggests the need for additional clear zone width, or whenever the radius of the curve is less than 2,860 feet, the design speed is 55 mph or greater, and the curve occurs on a normally tangent alignment (one where the

curve is preceded by a tangent more than a mile in length).

The clear zone along an urban section may contain minor obstructions (traffic signs, mailboxes, etc.). In addition, along lower (<40 mph design speed) urban roadways, larger objects designed to "break-away" when struck by a vehicle may also be located within the clear zone (light poles, cast-iron fire hydrants, etc.). All objects, however, should be kept free from the object setback zone as described in the next section.

L. Object Setback

Like clear zone, object setback is intended to provide an area adjacent to the roadway that is clear of obstructions. However, the purpose of the object setback is to provide an operational clearance to increase driver comfort and avoid a negative impact on traffic flow. It also improves aesthetics, provides an area for snow storage and, in areas with curbside parking, provides a clear area to open car doors.

As shown in [Section 5C-1](#), [Tables 5C-1.01](#) and [5C-1.02](#), minor obstructions and larger "breakaway" objects may be located in the clear zone on lower speed roadways (<40 mph design speed), but must be kept free from the object setback. Mailboxes constructed and installed according to US Postal Service regulations, including breakaway supports, may be located within the object setback area.

Additional object setback (typically 3 feet), as measured from the back of curb, may be required around radii at intersections and driveways in order to provide sufficient clearance to keep the overhang of a truck from striking an object.

M. Border Area

Border area is the area between the roadway and the right-of-way line and is sometimes referred to as the "parking" in urban areas. The grade for the border area is normally 1/2 inch per foot. The border area between the roadway and the right of way line should be wide enough to serve several purposes including provision of a buffer space between pedestrians and vehicular traffic, sidewalk space, and an area for both underground and above ground utilities such as storm sewer, traffic signals, parking meters, and fire hydrants. The border area also provides snow storage and aesthetic features such as grass or other landscaping features. The border width ranges from 14 to 16 feet, including the sidewalk width. Traffic signals, utility poles, fire hydrants, and other utilities should be placed as far back of the curb as practical for safety reasons. Breakaway features should be built when feasible and as an aid to safety considerations.

Table 5C-2.02: Preferred Border Area

Street Classification	Border Area Width (feet)
Major/minor arterial	16
Collector	14.5
Local streets	14

N. Curbs

1. **Curb Offset:** The curb offset is measured from the back of curb to the edge of the lane. The curb offset increases driver comfort and roadway safety. The presence of the curb, and potential vehicle damage and loss of control resulting from striking the curb, causes drivers to move away from the curb, reducing the effective width of the through lane. Due to this driver reaction, and to accommodate the flow of drainage and intake structures, an offset between the curb and the edge of the traveled way is provided.

The curb offset widths specified in [Section 5C-1](#), [Tables 5C-1.01](#) and [5C-1.02](#) do not necessarily indicate the width of the curb and gutter or the location of a longitudinal joint; however, the width of the curb and gutter can affect the required width of the curb offset. The presence of a longitudinal joint near the curb (gutterline jointing) can be a limiting factor for usable lane width as some drivers are uncomfortable driving on or near the joint line. This is especially true for HMA roadways with PCC curb and gutter. For pavements with a longitudinal joint line near the gutter, the curb offset should be equal to or greater than the width of the curb and gutter section. In addition, grates and special shaping for curb intakes and depressions for open-throat intakes should be located within the curb offset width and should not encroach into the lane.

2. **Curb and Gutter:** Typically, a curb should be 6 inches high, 6 inches wide. Where curb and gutter is used, the standard gutter width is 2 feet, 6 inches. When a gutter is used, it should not be included in the travel lane width. If the design speed is 40 mph or below, an 8 inch curb may be used for certain arterial and collector streets. For design speeds greater than 40 mph, a 1 foot wide, 6 inch high sloped curb with up to a 2 foot offset may be used. Where a gutter is used, its width is considered part of the curb offset width.

O. Parking Lane

Where curbed sections are used, the curb offset width may be included as part of the parking lane.

1. Parking lanes are not allowed on arterial streets.
2. Although on-street parking may impede traffic flow, parallel parking may be allowed by the Jurisdiction on urban collectors where sufficient street width is available to provide parking lanes.
3. Parking lane width determinations should include consideration for the potential use of the lane as a through or turn lane for moving traffic either during peak hours or continuously. If this potential exists, additional parking width should be provided.

P. Cul-de-sacs

A local street open at one end only should have a cul-de-sac constructed at the closed-end. The 2018 International Fire Code stipulates a minimum cul-de-sac radius of 48 feet however some jurisdictions allow lesser radii due to the size of their fire apparatus. The minimum radius for cul-de-sacs is 45 feet, which may be increased in commercial areas or if significant truck traffic is anticipated. The border area around the cul-de-sac should be the same as the approach street. The transition radius with the approach street will be 50 feet for residential streets and 75 feet for commercial and industrial streets.

The length of a cul-de-sac determines how many people are impacted by maintenance operations, traffic accidents, and other incidences that may stop traffic flow. Many Iowa cities limit the length of a cul-de-sac to 500 to 600 feet. Studies indicate the longer the cul-de-sac, the higher the vehicular

speeds along it. The 2018 edition of the International Fire Code recommends the length of the cul-de-sac be less than 750 feet unless additional steps such as intermediate turnarounds are implemented. ITE, the Urban Land Institute, and ASCE indicate cul-de-sacs should be less than 1,000 feet long or the length that generates less than 200 trips per day according to the adjacent land use. For single family dwellings that generate 8 to 10 trips per day, the 200 trips per day would be produced by about 20 parcels.

Consider building cut through sidewalks or shared use paths at the closed end of the cul-de-sac to improve pedestrian and bicyclist connections to surrounding neighborhood or land uses. The cut through sidewalk or shared use paths is likely to reduce vehicular trips by encouraging walking and bicycling.

Q. Shoulder Width

Shoulders accommodate stopped vehicles, emergency use, and provide lateral support of the subbase and pavement. In some cases, the shoulder can accommodate pedestrians when no sidewalks are present and bicyclists. Where no curb and gutter is constructed a soil, granular, or paved shoulder will be provided. When pedestrians and/or bicyclists are expected to use the shoulder, the shoulder should be paved. Refer to [Section 12B-3](#) for guidance on paved shoulder widths. The *AASHTO Guide for the Planning, Design and Operation of Pedestrian Facilities* presents appropriate methods for accommodating pedestrians on paved shoulders. Where shoulders are designated as the pedestrian access route, shoulder must also meet accessibility requirements.

Desirably, a vehicle stopped on the shoulder should clear the pavement edge by 2 feet. This preference has led to the adoption of 10 feet as the desirable shoulder width that should be provided along high volume facilities. In difficult terrain and on low volume highways, usable shoulders of this width may not be practical.

Where roadside barriers, walls, or other vertical elements are used, the graded shoulder should be wide enough that these vertical elements can be offset a minimum of 2 feet from the outer edge of the usable shoulder. It may be necessary to provide a graded shoulder wider than used elsewhere on the curved section of a roadway or to provide lateral support for guardrail posts and/or clear space for lateral dynamic deflection required by the particular barrier in use. On low volume roads, roadside barriers may be placed at the outer edge of the shoulder; however, a minimum of 4 feet should be provided from the traveled way to the barrier.

R. Intersection Radii and Right Turning Vehicle Speeds

Vehicle turning movements affect operations and safety at an intersection and driveways – especially the safety of pedestrians and bicyclists. It is important to consider the size of vehicles that will reasonably be expected to move through the intersection, the frequency of these movements, and any local jurisdictional policies for lane encroachment. For roadways where the most common vehicle is a passenger car, delivery vehicle, or single unit truck, designing intersections to easily accommodate larger vehicles with large turning radii can negatively affect crossing distances, exposure to conflicts, the speed of turning vehicles, the severity of crashes, and amount of right-of-way needed for the intersection. Similarly, using a smaller design vehicle at intersections that are regularly used by larger vehicles should also be avoided because frequent operational challenges may occur, may lead to encroachment beyond the edge of pavement or curb line, and can lead to damage to infrastructure such as curb ramps, signs, or poles. The following sections describe the process for selecting the appropriate design vehicle and intersection turning radii.

1. **Selecting Intersection Design and Control Vehicles:** Typical design and control vehicles, described below, are shown in Table 5C-2.03 for different roadway classifications. Refer to the current AASTHO Green Book for turning templates.
 - a. **Intersection Design Vehicle (IDV):** The design vehicle is the least maneuverable vehicle that routinely uses the street. Designers use a design vehicle to determine corner radii at intersections and should use this vehicle when completing analysis with turning analysis software. If an intersection includes a bus route where buses make turns, an appropriately-sized bus may be used as the design vehicle. In many jurisdictions, a standard box truck (SU-30) or a school bus is the default design vehicle. The current AASHTO Green Book provides turning templates for a variety of design vehicles. The Iowa DOT has an Iowa truck vehicle that can be used to check the proposed radii for truck routes.
 - b. **Intersection Control Vehicle (ICV):** The control vehicle is an infrequent but necessary user of the street. The control vehicle for intersection design is often a moving truck or a fire truck; there may be local jurisdictional policies to decide which control vehicle to use. The control vehicle can be assumed to use other traversable parts of an intersection, including across centerlines, also known as encroachment. Encroachment is the ability for a vehicle to use space outside of its designated travel lane, but within the roadway, to navigate a turning movement. Encroachment does not include tracking over curbs or onto the sidewalk area. Encroachment into opposing traffic lanes is discouraged on all roadways. Consult the local jurisdiction's current fire official to ensure the design meets emergency response needs.

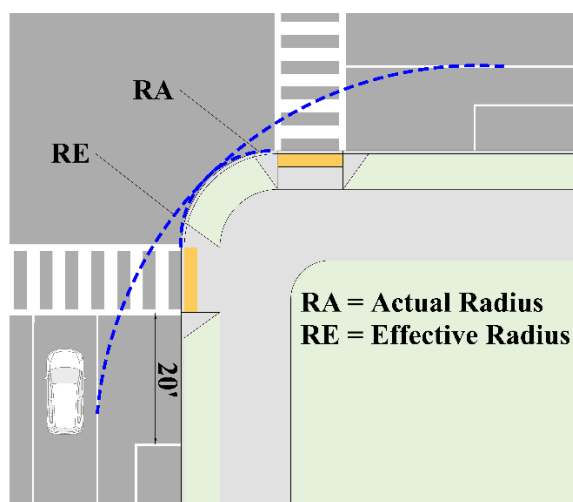
Figure 5C-2.10: Degrees of Encroachment

		DEPARTURE		
		1 no encroachment	2 encroachment into adjacent lane in same direction <small>(note: Same as 1 for single lane departure)</small>	3 encroachment into opposing lane
APPROACH	A no encroachment	 Degree of Encroachment = A1	 Degree of Encroachment = A2	 Degree of Encroachment = A3
	B encroachment into adjacent lane in same direction <small>(note: Same as A for single lane approach)</small>	 Degree of Encroachment = B1	 Degree of Encroachment = B2	 Degree of Encroachment = B3
	C encroachment into opposing lane	 Degree of Encroachment = C1	 Degree of Encroachment = C2	 Degree of Encroachment = C3

Source: Adapted from WisDOT Facility Design Manual

3. **Actual and Effective Curb Radius:** Two distinct radii need to be considered when designing street corners. The first is the actual radius of the street corner itself, and the second is the effective turning radius of the selected design vehicle or control vehicle, see Figure 5C-2.11. The effective turning radius is the radius needed for a turning vehicle to clear any adjacent parking lanes and/or to align itself with its new travel lane. Using an effective turning radius allows a smaller curb radius than would be required for the motorist to turn from curb lane to curb lane. It is critical that encroachment does not include tracking over the curbs or into the sidewalk area.

Figure 5C-2.11: Actual Corner Radius vs. Effective Turning Radius



Source: Adapted from NACTO Urban Street Design Guide

4. **Turning Vehicle Design Speed:** At both signalized and unsignalized intersections (including roundabouts), steps should be taken to ensure that turning speeds are kept low and that sight distance is not compromised for pedestrians, bicyclists, or motorists. While performing swept path analyses, the maximum recommended turning speed of the design and control vehicle is 10 mph. See [Section 5M-1](#) for turning speeds when pedestrians and bicyclists are present.
5. **Selecting Intersection or Driveway Corner Radii:** Designers should strive to provide the appropriate corner radius for the given IDV and ICV, target turning speeds, acceptable lane encroachment, number of receiving lanes, and effective pavement width. In addition to discouraging higher turning speeds, smaller corner radii are preferred in order to better align curb ramps with pedestrian paths of travel and shorten crossing distances.

S. Pavement Thickness

Refer to [Section 5F-1](#) for pavement thickness determination and design.

T. References

American Association of State Highway and Transportation Officials (AASHTO). *A Policy on Geometric Design of Highways and Streets* (referred to as “Green Book”). Washington, DC. 2018.

American Association of State Highway and Transportation Officials (AASHTO). *Guide for the Planning, Design, and Operation of Pedestrian Facilities*. Washington, DC. 2004.

American Association of State Highway and Transportation Officials (AASHTO). *Roadside Design Guide*. 3rd ed. Washington, DC. 2006.

Des Moines Area Metropolitan Planning Organization (MPO). *Des Moines Area Daily Directional Capacities at Level of Service D*. Des Moines. 2000.

Federal Highway Administration (FHWA). *Achieving Multimodal Networks: Applying Design Flexibility and Reducing Conflicts*. Washington, DC. 2016.

National Association of City Transportation Officials, (NATCO). *Urban Street Design Guide*. New York, New York. 2013.

Wisconsin Department of Transportation (WisDOT). *Facilities Development Manual (FDM)*. Madison, WI. 2022.