Chapter Twenty-eight
INTERSECTIONS AT-GRADE

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Chapter Twenty-eight
INTERSECTIONS AT-GRADE

Chapter Twenty-eight discusses the geometric design of at-grade intersections including intersection alignment and profile, right-turn designs, turning lanes, turning roadways, intersection sight distance, channelization, median openings and approaches. The intersection is an important part of the highway system. The operational efficiency, capacity, safety and cost of the system depend largely upon its design, especially in urban areas. The primary objective of intersection design is to reduce potential conflicts between vehicles, bicycles and pedestrians while providing for the convenience, ease and comfort of those traversing the intersection.

28.1 DEFINITIONS

1. Approach. A road providing access from a public way to a highway, street, road or to an abutting property.

2. Begin Curb Return (BCR). The point along the mainline pavement edge where the curb return of an intersection meets the tangent portion.

3. Channelization. The directing of traffic through an intersection by the use of pavement markings (e.g., striping, raised reflectors), medial separators or raised islands.

4. Comfort Criteria. Criteria that is based on the comfort effect of change in vertical direction in a sag vertical curve because of the combined gravitational and centrifugal forces.

5. Corner Island. A raised or painted island to channel the right-turn movement.

6. Design Vehicle. The vehicle used to determine turning radii, off-tracking characteristics, pavement designs, etc., at intersections.

7. End Curb Return (ECR). The point along the minor roadway pavement edge where the curb return of an intersection meets the tangent portion.

8. Face of Curb. A vertical line drawn from the intersection point of the vertical portion of the curb and the horizontal slope of the gutter or pavement.

9. Grade Separation. A crossing of two highways, or a highway and a railroad, at different levels.

10. Interchange. A system of ramps in conjunction with one or more grade separations, providing for the movement of traffic between two or more roadways on different levels.

11. Intersection. The general area where two or more highways join or cross at grade.

12. Intersection Sight Distance (ISD). The sight distance required within the corners of intersections to safely allow a variety of vehicular access or crossing maneuvers based on the type of traffic control at the intersection.

13. Islands. Channelization (raised or flush) in which traffic passing on both sides is traveling in the same direction.

14. Landing Area. The area approaching an intersection for stopping and storage of vehicles.

15. Medial Separator. Channelization that separates opposing traffic flows, alerts the driver to the cross road ahead and regulates traffic through the intersection.

16. Median Opening. Openings in the median (raised or depressed) on divided facilities that allow vehicles to cross the facility or to make a turn.

17. No Control Intersection. An intersection where none of the legs are controlled by a traffic control device.

18. Return. The circular segment of curb at an intersection that connects the tangent portions of the intersecting legs.

19. Roundabout. A circulatory road at an intersection that redirects traffic around a central island.

20. Signalized Intersection. An intersection where all legs are controlled by a traffic signal.

21. Stop-Controlled Intersection. An intersection where one or more legs are controlled by a stop sign.

22. Turn Lane. An auxiliary lane adjoining the through traveled way for speed change, storage and turning.

23. Turning Roadway. A channelized roadway, created by an island, connecting two legs of an at-grade intersection. Interchange ramps are not considered turning roadways.

24. Turning Template. A graphic representation of a design vehicle's turning path depicting various angles of turns for use in determining acceptable turning radii designs.

25. Yield-Controlled Intersection. An intersection where one or more legs are controlled by a yield sign.

28.2 GENERAL DESIGN CONTROLS

28.2.1 Capacity and Level of Service

A capacity analysis should be conducted before performing the detailed design of any intersection. This analysis will influence several geometric design features including the number of approach lanes, turning lanes, lane widths, channelization and number of departure lanes. The designer should select a level of service and future design year, typically 20 years from the construction completion date. If the intersection is within the limits of a longer project, the design year for the intersection will be the same as that for the project.

Level-of-service recommendations are provided in the geometric design tables in Chapter Twelve of the Montana Road Design Manual. Once the level of service and design traffic volumes are determined, the designer should use the Highway Capacity Manual for the detailed capacity analysis; see Chapter Thirty.

28.2.2 Design Vehicle

The basic design vehicles which may be used for intersection designs are:

1. P — Passenger car and light panel and pickup trucks.
2. SU — Single-unit truck or small bus.
3. CITY-BUS — Transit bus.
4. WB-50 (WB-15) — Semitrailer combination with an overall wheelbase of 50 ft (15.2 m).
5. WB-67 (WB-20) — Semitrailer combination with a 53 ft (16.2 m) trailer.
6. WB-100T (WB-30T) — Semitrailer combination with three 28 ft (8.5 m) trailers.
7. MH — Recreational vehicle, motor home.

The turning characteristics of the applicable design vehicle are used to test the adequacy of an existing or proposed design at an intersection. The turning characteristics can be checked with turning templates or a computer-simulated turning template program (e.g., AutoTURN).

28.2.3 Intersection Spacing

If practical, avoid short distances between intersections because they tend to impede traffic operations. For example, if two intersections are close together and require signalization, they may need to be considered as one intersection for signalization purposes. To operate safely, each leg of the intersection may require a separate green phase, thereby greatly reducing the capacity for both intersections. To operate efficiently, signalized intersections should desirably be ¼ mi (400 m) apart. Short spacing between intersections may hinder or even restrict effective left-turn movements. Where practical, realign the roadways to form a single intersection.

In addition, also avoid short gaps between opposing “T” intersections. Drivers tend to encroach into the opposing lanes (corner cutting) so that they can make their turning maneuvers in one movement. In general, all new intersections should preferably be at least 400 ft (120 m) apart.

28.2.4 Intersection Alignment

28.2.4.1 Horizontal Curves

Preferably, an intersection between two roadways should be on tangent sections. When a minor road intersects a major road on a horizontal curve, the geometric design of the intersection becomes significantly more complicated, particularly for sight distance, turning movements, channelization and superelevation.

28.2.4.2 Angle of Intersection

Desirably, roadways should intersect at or as close to 90° as practical. Skewed intersections are undesirable for several reasons:

1. Vehicular turning movements become more restricted.
2. The accommodation of large trucks for turning may require additional pavement and channelization.
3. The exposure time for vehicles and pedestrians crossing the main traffic flow is increased.
4. The driver’s line of sight for one of the sight triangles becomes restricted.

The intersection angle should not exceed 30° from perpendicular. Intersections with a skew greater than 30° from perpendicular must be reviewed and documented. For existing intersections, it will rarely be warranted to realign the intersection if its skew is within 30° of perpendicular. Where skew angles greater than 30° are present, the intersection may require geometric improvements (e.g., realignment, auxiliary lane, greater corner sight distance). Figure 28.2A illustrates various angles of intersection and potential improvements that can be made to the alignment.

28.2.5 Intersection Profile

28.2.5.1 Gradient

The "landing area" is that portion of intersecting highways, local roads and public and private approaches that will be used for the storage of stopped vehicles. Desirably, the landing area will slope downward from the intersection on a gradient not to exceed 3%. An upward sloping landing area should be avoided, if practical. However, if site constraints warrant, the landing area may slope upward from the intersection on a gradient not to exceed 3%. At a minimum, the landing area should be 75 ft (25 m) for public roads and 25 ft (7.5 m) for other facilities.

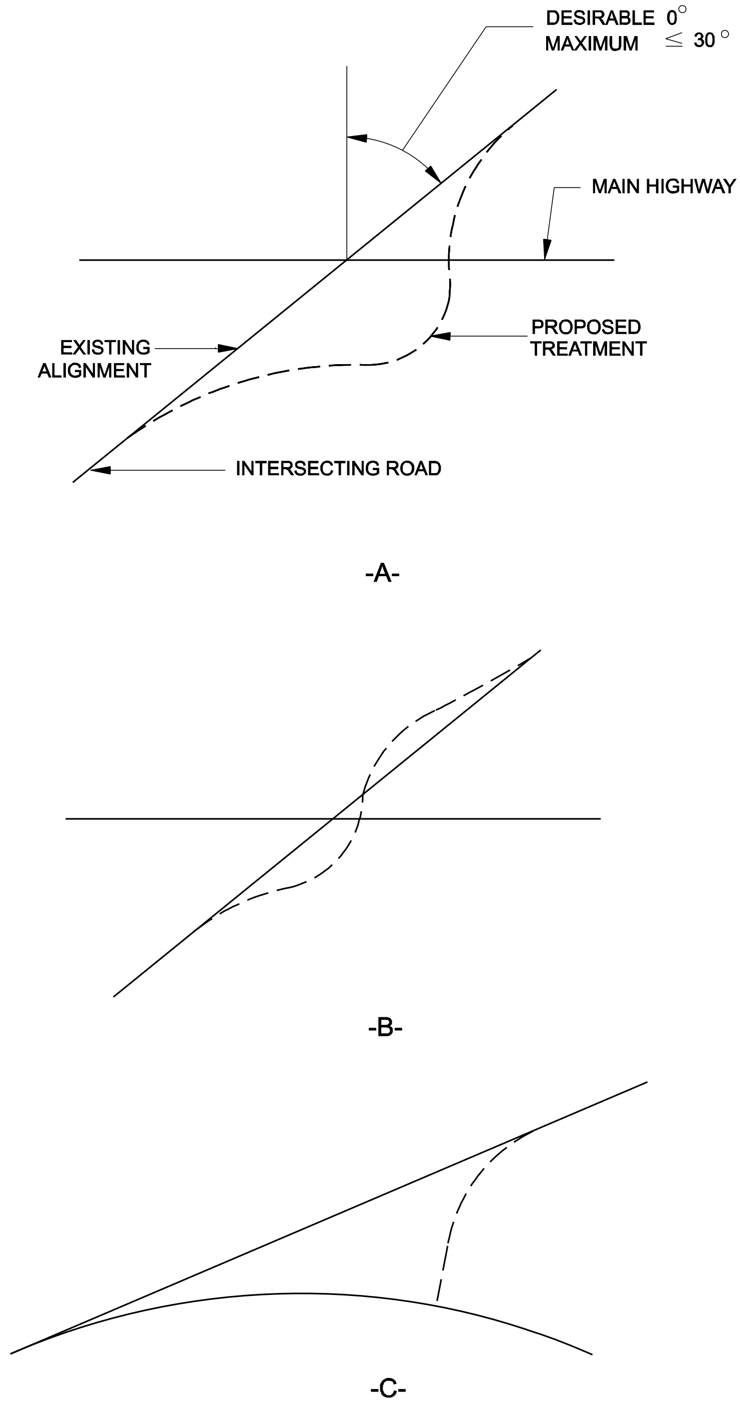
The gradient of the approach beyond the 25 ft (7.5 m) landing should not exceed 6% for public or private approaches unless site constraints preclude its use. The gradient of farm field approaches should not exceed 10%. Use of steeper approach slopes does not require a design exception, but must be documented in the Alignment Review Report or Plan-in-Hand Report.

28.2.5.2 Cross Slope Transitions

One or both of the roadways approaching the intersection may need to be transitioned (or warped) to match or coordinate the cross slope and grade at the intersection. The designer should consider the following:

1. Stop Controlled. When the minor road is stop controlled, the profile gradeline and cross slope of the major road will normally be maintained through an intersection, and the cross slope of the stop-controlled leg will be transitioned to match the major road grade.

2. Signalized Intersection. At signalized intersections, or potential signalized intersections, the cross slope of the minor road will typically be transitioned to meet the grade of the major road. If both intersecting roads have approximately equal importance, the designer may want to consider transitioning both roadways to form a plane section through the intersection. Where compromises are



Note: Check the superelevation on the horizontal curve.

TREATMENTS FOR SKEWED INTERSECTIONS

Figure 28.2A

necessary between the two major roadways, the smoother riding characteristics should be provided for the roadway with the higher travel speeds.

3. Transition Distance. In rural areas, transitioning from the normal crown to a warped section should be accomplished in a distance of 50 ft (15 m). For urban areas, the 50 ft (15 m) transition length is also desirable but, at a minimum, the transition may be accomplished within the curb return; see Figure 28.2B.

28.2.5.3 Vertical Profile

Where the profile of the minor road is adjusted to meet the major road, this will result in angular breaks for traffic on the minor road if no vertical curve is inserted. The following options are presented in order from the most desirable to the least desirable; see Figure 28.2C:

1. Vertical Curves (SSD). Desirably, vertical curves will be used through an intersection which meet the criteria for stopping sight distance as described in Chapter Twenty-six. For stop-controlled legs, design the approach landing vertical curve with a 30 mph (50 km/h) design speed; at free-flowing legs and at all legs of a signalized or proposed future signalized intersection, use the design speed of the roadway to design the vertical curve. The grades of the tangents for the vertical curve are the grade of the landing area (G_1) and the profile grade of the minor roadway (G_2); see Figure 28.2C. The Vertical Point of Tangency (VPT) will be located at the end of the landing 75 ft (25 m) from the paved shoulder of the mainline). The VPT can be shifted onto the landing area if the gradient of the landing does not exceed 3%.

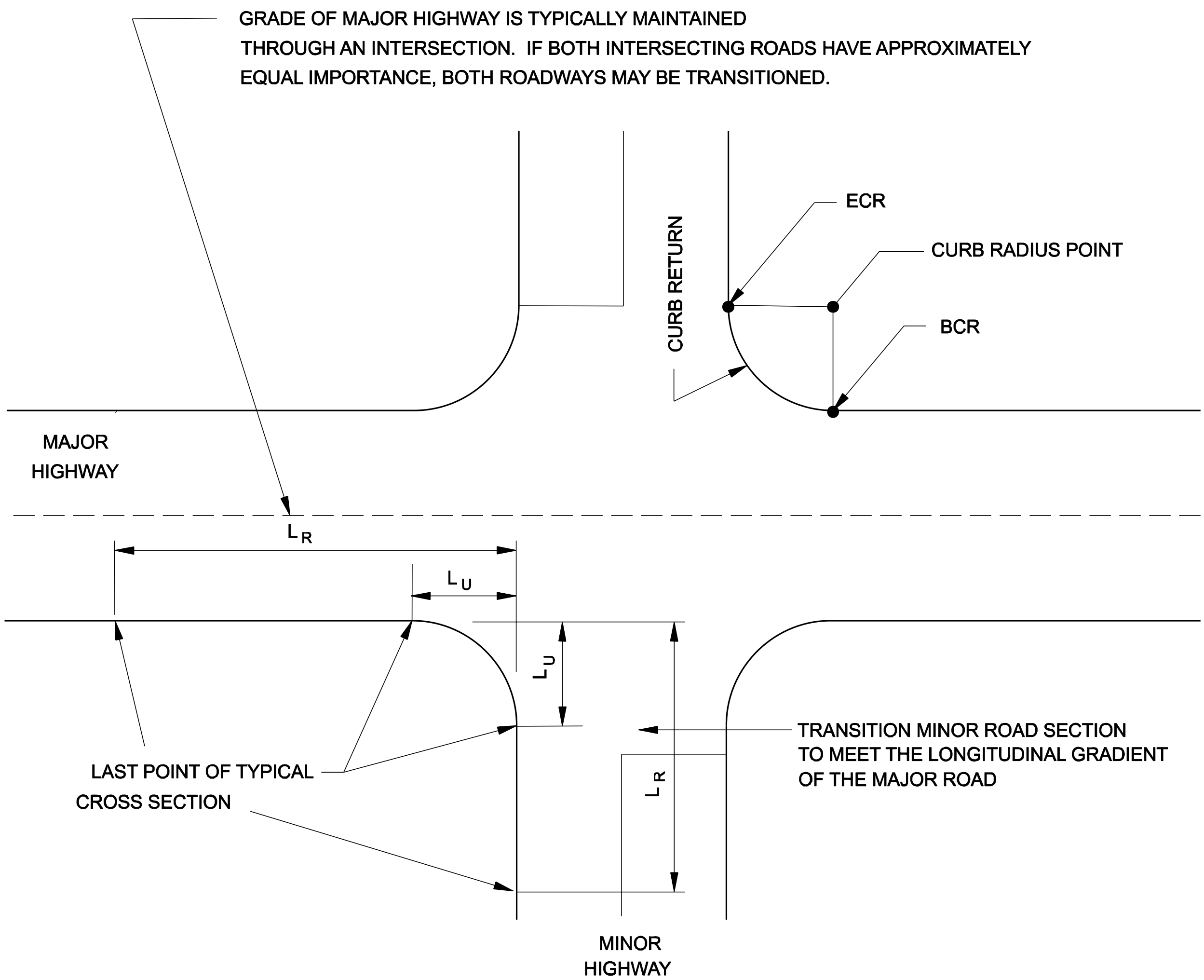
2. Sag Vertical Curves (Comfort). For sag vertical curves, the next most desirable option is to design the sag to meet the comfort criteria. The length of vertical curve can be determined as follows:

$$L = \frac{AV^2}{46.5} \qquad \text{(US Customary) (Equation 28.2-1)}$$

$$L = \frac{AV^2}{395} \qquad \text{(Metric) (Equation 28.2-1)}$$

Where:

L = length of vertical curve, ft (m)
A = algebraic difference between grades, %
V = design speed, mph (km/h)

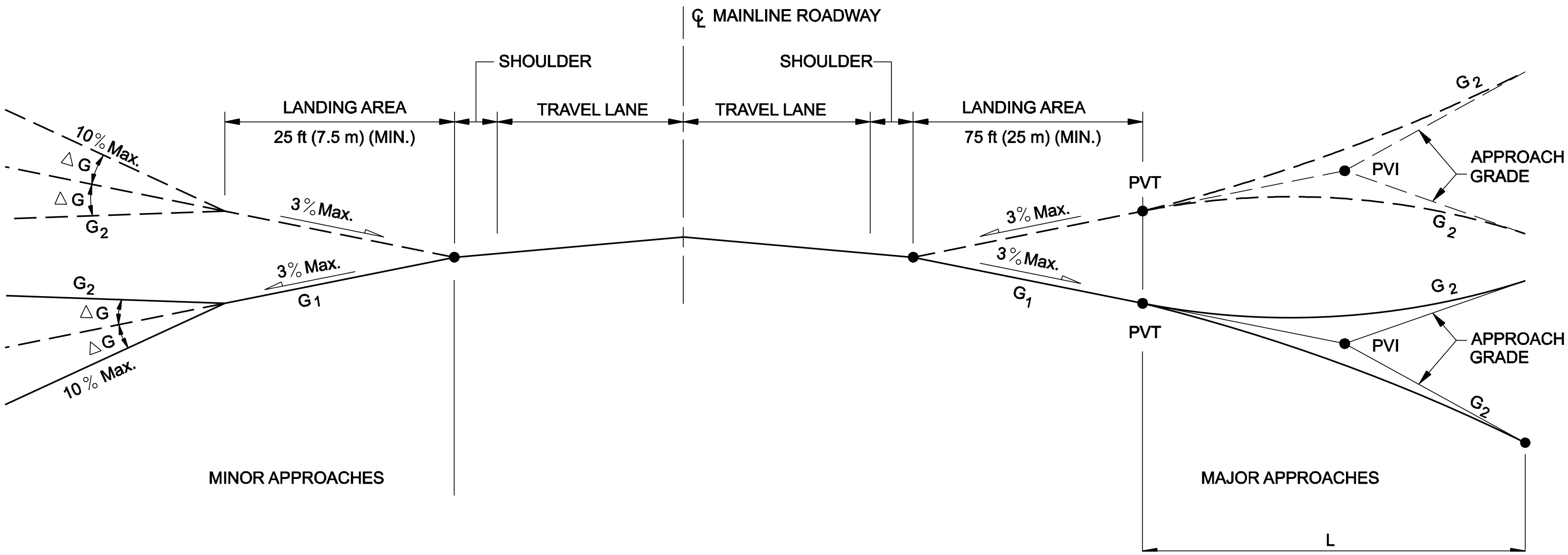


L_R = TRANSITION LENGTH FOR RURAL HIGHWAYS 50 ft (15 m)

L_U = TRANSITION LENGTH FOR URBAN HIGHWAYS

PAVEMENT TRANSITIONS THROUGH INTERSECTIONS

Figure 28.2B



Notes:

1. *At signalized intersections, the most desirable rotation option will be to transition the cross slopes of all approach legs into a plane section through the intersection.*

2. *If practical, the gradient of the landing area where vehicles may be stored should not exceed 3%.*

3. *See Figure 28.2D for maximum allowable ΔG's.*

4. *Actual field conditions will determine the final design.*

VERTICAL PROFILES OF INTERSECTING ROADS

Figure 28.2C

3. Vertical Curves (Minimum Comfort). Under restricted conditions where a design based on SSD or comfort is not practical, vertical curves at intersection approaches may be based on the following formulas:

$K = (0.1V)^2$ (Sag Curves) (US Customary) (Equation 28.2-2)
$K = (0.034V)^2$ (Sag Curves) (Metric) (Equation 28.2-2)

$K = (0.07V)^2$ (Crest Curves) (US Customary (Equation 28.2-3)
$K = (0.024V)^2$ (Crest Curves) (Metric) (Equation 28.2-3)

$L = KA$ (Equation 28.2-4)

Where:

K = the horizontal distance in feet (meters) needed to produce a 1% change in the gradient along the curve

A = algebraic difference between the two tangent grades, %

V = design speed, mph (km/h)

L = length of vertical curve, ft (m)

4. Angular Breaks. Angular breaks between the landing area and the approach gradient are typically used on minor approaches; see Figure 28.2C. For major approaches, it may be impractical to provide vertical curves on the approaches under some restricted conditions; i.e., angular breaks are necessary through the intersection. Figure 28.2D provides the maximum allowable angular breaks for various design speeds. Where angular breaks are used, the minimum distance between successive angle points should be at least 15 ft (5 m).

28.2.5.4 Intersection Sight Distance

The designer needs to consider the effect the intersection profile and alignment will have on intersection sight distance. Landings with steep upgrades may put the driver's eye below or in line with roadway appurtenances (e.g., guardrail, signs). Also, large skewed intersections will require the driver to look back over their shoulder. For more information on intersection sight distance, see Section 28.9.

US Customary		
Design Speed	Crest Vertical Curves (ΔG)	Sag Vertical Curves (ΔG)
20 mph	7.0%	4.3%
25 mph	5.4%	2.7%
30 mph	3.8%	1.9%
35 mph	2.8%	1.4%
40 mph	2.1%	1.0%
45 mph	1.7%	0.8%
50 mph	1.4%	0.7%
55 mph	1.1%	0.5%
60 mph	1.0%	0.4%

Metric		
Design Speed	Crest Vertical Curves (ΔG)	Sag Vertical Curves (ΔG)
30 km/h	7.5%	4.8%
40 km/h	5.4%	2.7%
50 km/h	3.5%	1.7%
60 km/h	2.4%	1.2%
70 km/h	1.8%	0.8%
80 km/h	1.4%	0.7%
90 km/h	1.1%	0.5%
100 km/h	0.9%	0.4%

Notes:

1. *Design speed applies to the roadway with the angular break. Typically, this will be the minor roadway.*

2. *The angular break (ΔG) occurs between the landing area and approach roadway; see Figure 28.2C.*

MAXIMUM CHANGE IN GRADES WITHOUT A VERTICAL CURVE

Figure 28.2D

28.3 TURNING RADII

28.3.1 Types of Returns

At intersections, the designer must determine how best to accommodate right-turning vehicles. A design must be selected for the edge of pavement or curb lines, which may be one of the following types:

1. simple radius,
2. compound curve (2 or 3 centered), or
3. simple radius with entering and/or exiting taper(s).

Figure 28.3A illustrates these radii designs. Figure 28.3A also illustrates the effect the WB-67 (WB-20) design vehicle has on the various radii designs.

Each basic design type has its advantages and disadvantages. The simple radius is the easiest to design and construct and, therefore, it is the most common. However, the designer should also consider the benefits of compound radii or a simple radius with tapers. Their advantages as compared to simple radius designs include:

1. To accommodate a specific vehicle with no encroachment, a simple radius requires greater intersection pavement area than compound curves or a radius with tapers. For large vehicles, a simple radius is often an unreasonable design, unless a channelized island is used.

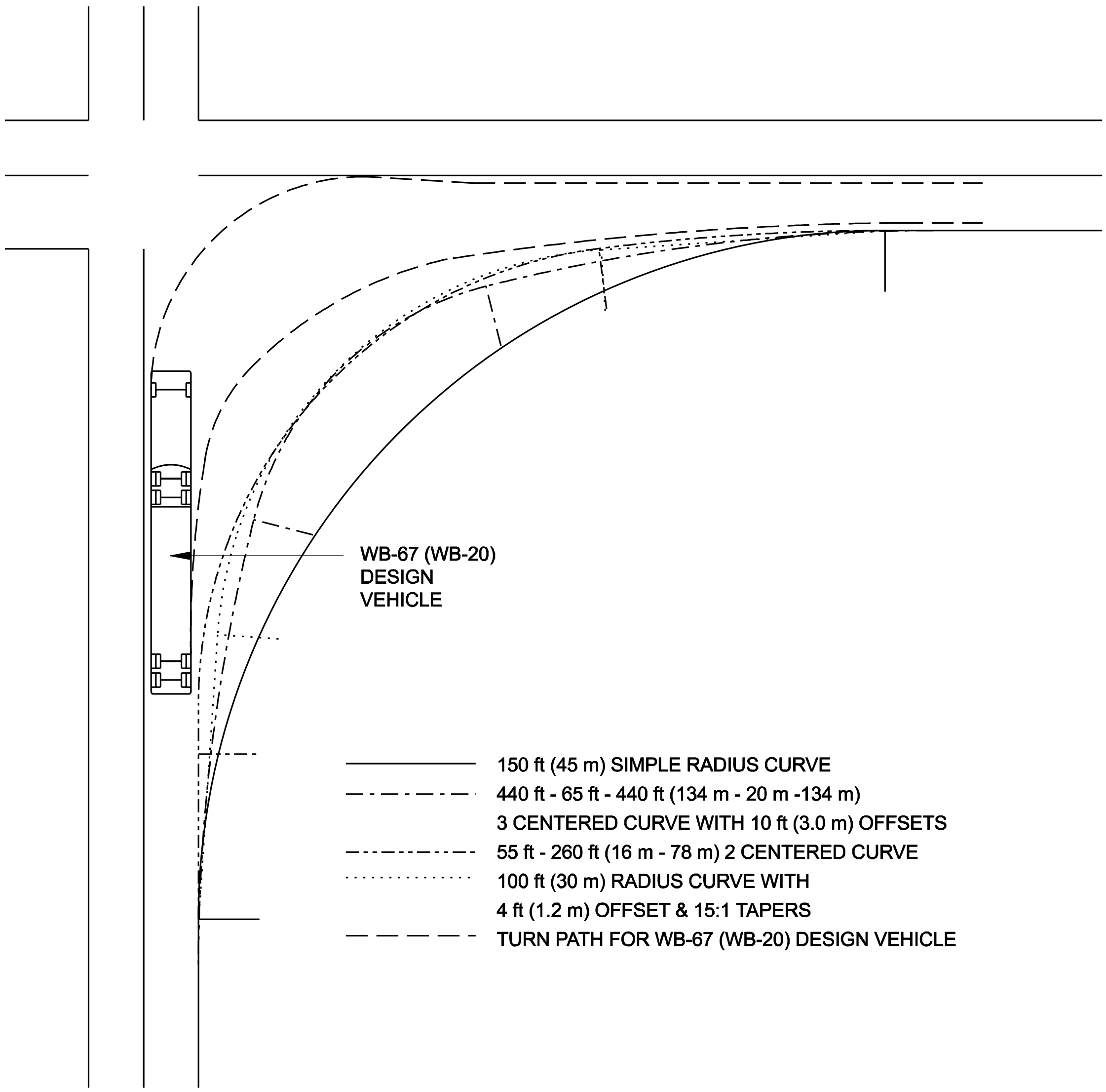
2. A simple radius results in greater distances for pedestrians to cross than compound curves or a radius with tapers.

3. For turns with angles greater than 90°, compound curves or a radius with tapers require less intersection area.

28.3.2 Right-Turn Designs

28.3.2.1 Design Considerations

The following presents several factors that need to be considered in determining the proper pavement edge/curb line for turns at intersections:



TYPICAL CURB RADII DESIGNS

Figure 28.3A

1. Design Vehicle. In general, select the design vehicle based on the largest vehicle that will use the intersection with some frequency. Section 28.2.2 lists the various design vehicles used by the Department. Figure 28.3B presents the suggested design vehicle based on the functional classification of the intersecting highways which the vehicle is turning from and onto. The designer should also check with the District Office to determine the typical vehicles which use a specific intersection.

2. Inside Clearance. The selected design vehicle will make the right turn while maintaining a minimum 2.0 ft (0.6 m) clearance from the pavement edge or face of curb.

3. Encroachment. The design vehicle should not be allowed to encroach into the opposing or adjacent lane while making the right turn.

4. Parking Lanes/Shoulders. At many intersections, parking lanes and/or shoulders will be available on one or both approach legs. This additional width will greatly ease the turning problems for large vehicles at intersections with small curb radii. Figure 28.3C illustrates the turning paths of several design vehicles with curb radii of 8 ft (2.4 m) and 25 ft (7.5 m). Parking should be restricted a sufficient distance from the intersection to allow the design vehicle to make the right or left turn. The designer should use the applicable turning template to determine this distance.

5. Pedestrians. The greater the turning radius, the farther pedestrians must walk across the roadway. This is especially important to disabled individuals. Larger radii also make it more difficult for drivers to see pedestrians. Therefore, the designer should consider pedestrian usage when determining the edge of pavement or curb line design.

6. Turning Design Types. Once the designer has determined the basic turning parameters (e.g., design vehicle, encroachment, inside clearance), it is necessary to select a type of turning design for the curb return or pavement edge that will meet these criteria and will fit the intersection constraints. Section 28.3.1 discusses the various radii designs used by MDT. The selection will be determined on a case-by-case basis.

7. Turning Template. To determine the design, the designer should apply a turning template for the selected design vehicle.

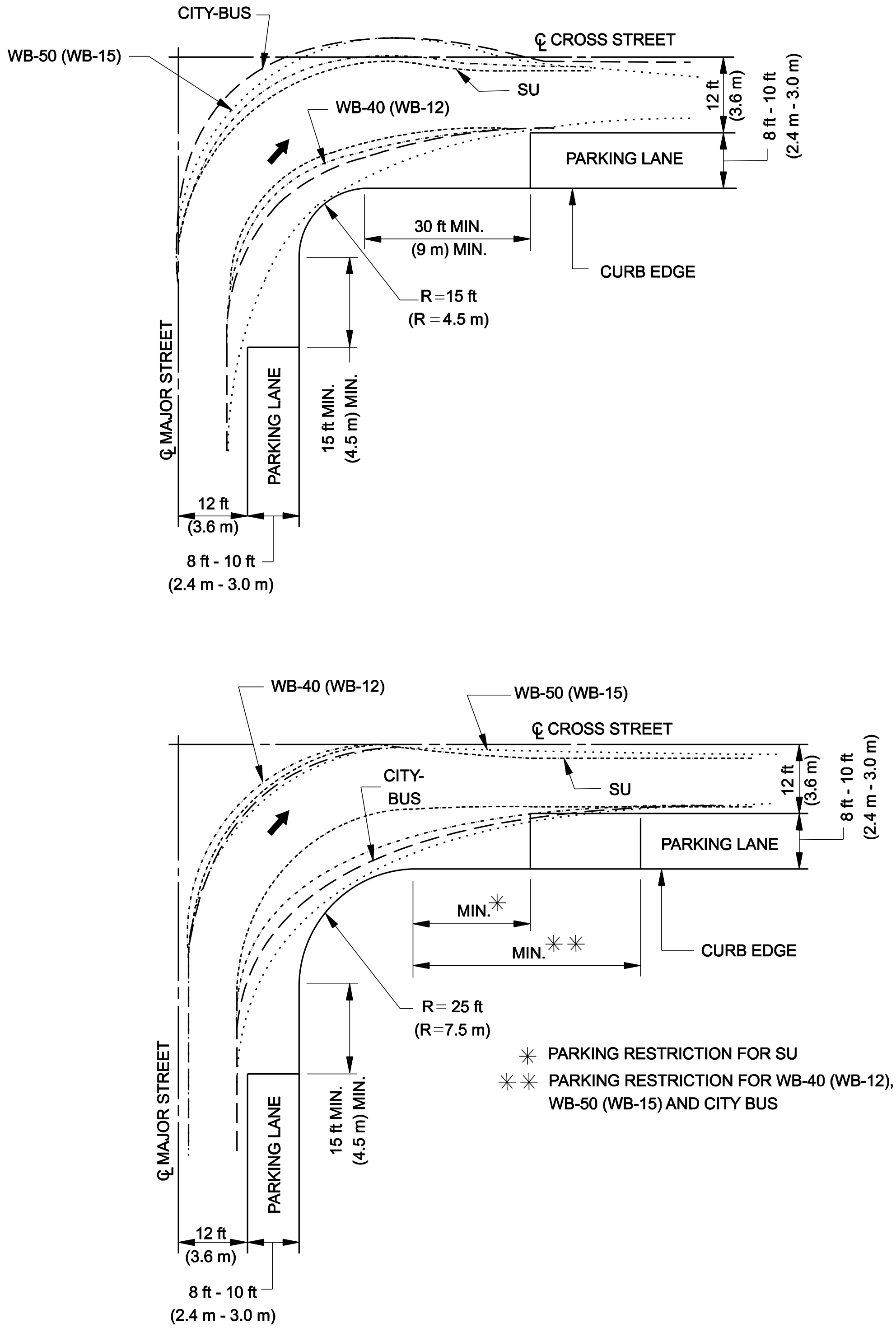
Figure 28.3D illustrates the many factors that should be evaluated in determining the proper design for right turns at intersections.

For Turn Made From	For Turn Made Onto	Suggested Design Vehicle (Rural)
Freeway Ramp	Other Facilities	WB-67 (WB-20)
Other Facilities	Freeway Ramp	WB-67 (WB-20)
Arterial	Arterial Collector Local	WB-67 (WB-20) WB-67 (WB-20*) Site Specific
Collector	Arterial Collector Local	WB-67 (WB-20*) WB-67 (WB-20*) Site Specific

***For urban intersections, the selected design vehicle will be site specific.**

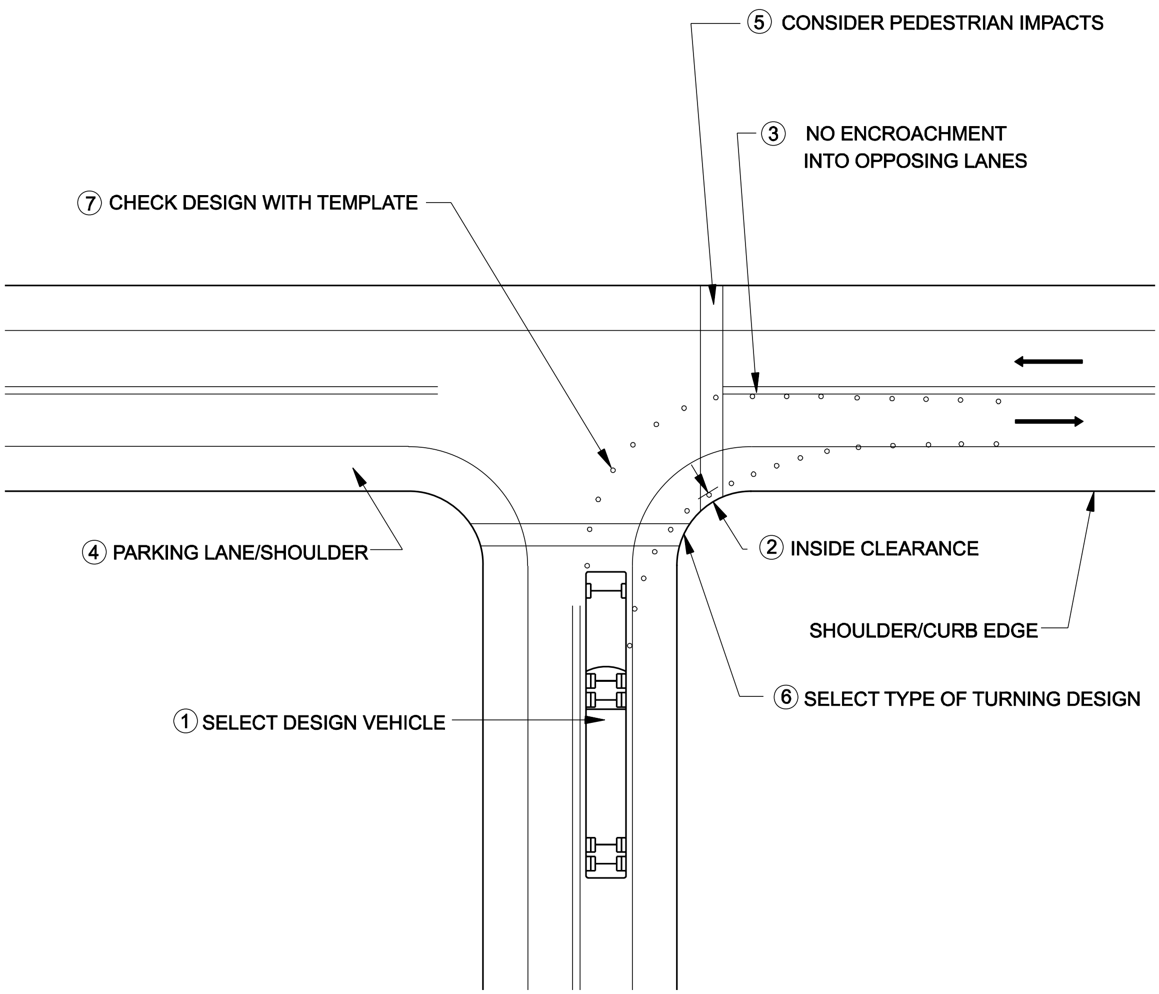
SUGGESTED DESIGN VEHICLE SELECTION
(Intersections)

Figure 28.3B



EFFECT OF CURB RADII AND PARKING ON TURNING PATHS

Figure 28.3C



Note: Numbers apply to the steps listed in Section 28.3.2.

TURNING RADII DESIGN SUMMARY

Figure 28.3D

28.3.2.2 Summary

In summary, when designing corner radii, consider the design vehicle, right-of-way, angle of intersection, number of pedestrians, width and number of lanes on the intersecting streets, and turning speeds. The following are additional guidelines to consider:

1. Minor Urban Intersections. Curb radii of 15 ft to 25 ft (4.5 m to 7.5 m) are adequate for passenger vehicles where there is a limited number of turning trucks or where there are parking lanes. Where the street has sufficient capacity to retain the curb lane as a parking lane, restrict parking for the appropriate distances from the intersection. Provide radii of 25 ft (7.5 m) or more at minor intersections on new construction and reconstruction projects.

2. Major Urban Intersections. Provide curb radii of 30 ft (9 m) or more at major cross streets so trucks may turn with minimal encroachment.

3. Trucks. Where large trucks and buses are frequent turning vehicles, provide radii of 40 ft (12 m) or more. These curb radii should preferably be 2- or 3-centered compound curves or simple curves with tapers.

4. Pedestrians. Coordinate radii designs with the crosswalk alignments or provide special designs for pedestrians (e.g., refuge islands, curb ramps).

28.3.3 Left-Turn Designs

Simple curves are typically used for left-turn designs. Occasionally, a 2-centered curve is desirable to accommodate the off-tracking of large vehicles provided the second curve has a larger radius.

The design values for left-turn control radii are a function of the design vehicle (off-tracking), angle of intersection, number of lanes and median width. For roadways intersecting at approximately 90°, radii of 40 ft to 90 ft (12 m to 27 m) will typically satisfy all controlling factors. The criteria for clearance offsets, encroachment, etc., listed in Section 28.3.2 are also applicable to left-turn designs.

28.4 AUXILIARY TURN LANES

28.4.1 Turn Lane Guidelines

28.4.1.1 Guidelines for Right-Turn Lanes

Exclusive right-turn lanes should be considered for the following situations:

1. at the free-flowing leg of any unsignalized intersection on a 2-lane urban or rural highway that satisfies the criteria in Figure 28.4A;
2. at the free-flowing leg of any unsignalized intersection on a high-speed, 4-lane urban or rural highway that satisfies the criteria in Figure 28.4B;
3. at any intersection where a capacity analysis determines a right-turn lane is necessary to meet the level-of-service criteria;
4. as a general rule, at any signalized intersection where the projected right-turning volume is greater than 300 vph and where there is greater than 300 vph per lane on the mainline; or
5. at any intersection where the crash trend involves right-turning vehicles.

28.4.1.2 Guidelines for Left-Turn Lanes

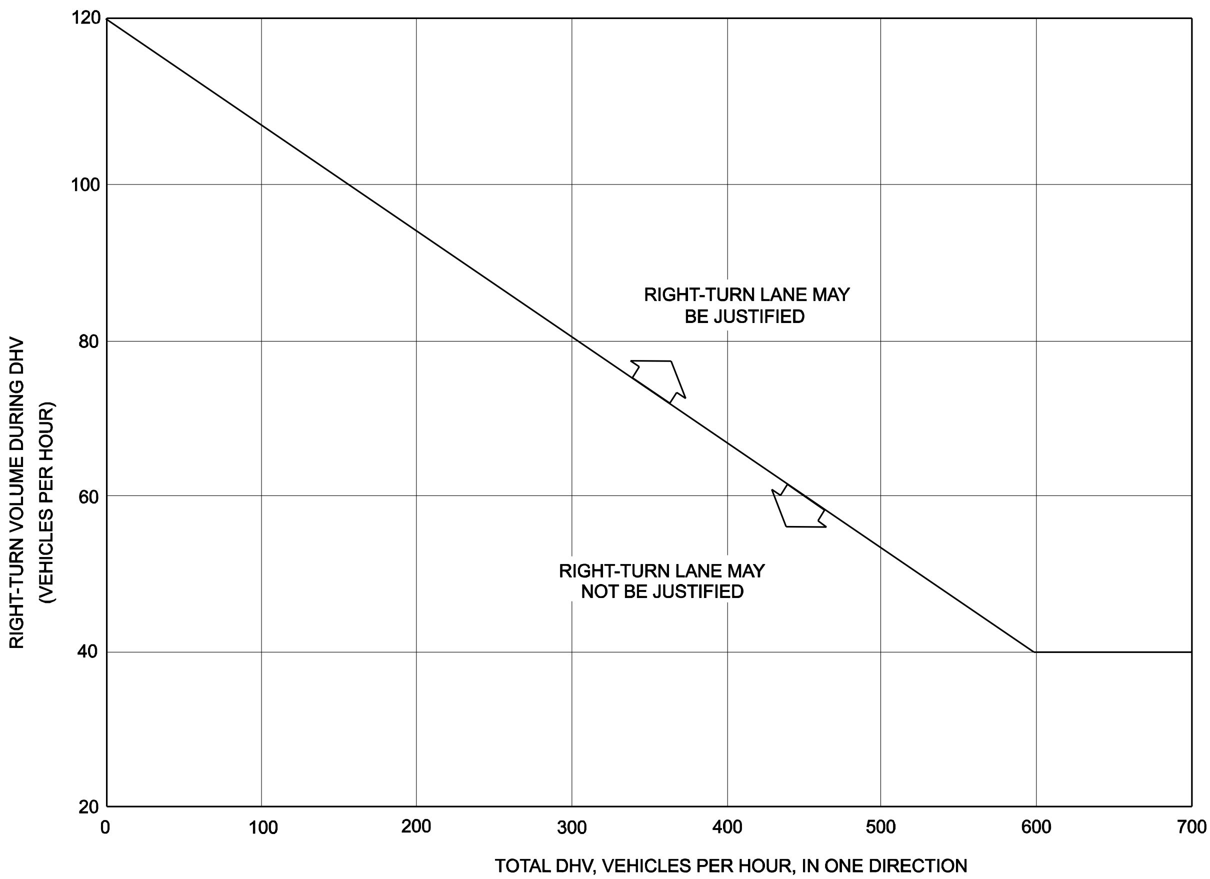
Exclusive left-turn lanes should be considered for the following situations:

1. at all public intersections on all multilane urban and rural highways, regardless of traffic volumes;
2. at the free-flowing leg of any unsignalized intersection on a 2-lane urban or rural highway that satisfies the criteria in Figures 28.4C, 28.4D, 28.4E or 28.4F;
3. at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria;
4. as a general rule, on the major roadway at any signalized intersection;
5. at high volume driveway approaches that satisfy the criteria in Figures 28.4C, 28.4D, 28.4E or 28.4F; or
6. at any intersection where the crash experience, traffic operations and/or sight distance restrictions (e.g., intersection beyond a crest vertical curve) indicate a significant conflict related to left-turning vehicles.

When establishing a left-turn lane, the designer needs to consider access to and from private properties on the legs to the intersection.

28.4.1.3 Sight Distance

When considering a right-turn lane on a through roadway, give specific attention to visibility on the side street. Decelerating vehicles in the auxiliary lane can create a moving sight obstruction from the side street approach. Proper placement of the stop bar on the side streets and lateral placement of right-turn lanes will allow a vehicle on the side street approach to see the approaching through traffic. Combination of medial separators and channelizing islands can be used to control proper placement of stopped and decelerating vehicles.



Note: For highways with a design speed below 50 mph (80 km/h) with a DHV < 300 and where right turns are > 40, an adjustment should be used. To read the vertical axis of the chart, subtract 20 from the actual number of right turns.

<u>Example</u>

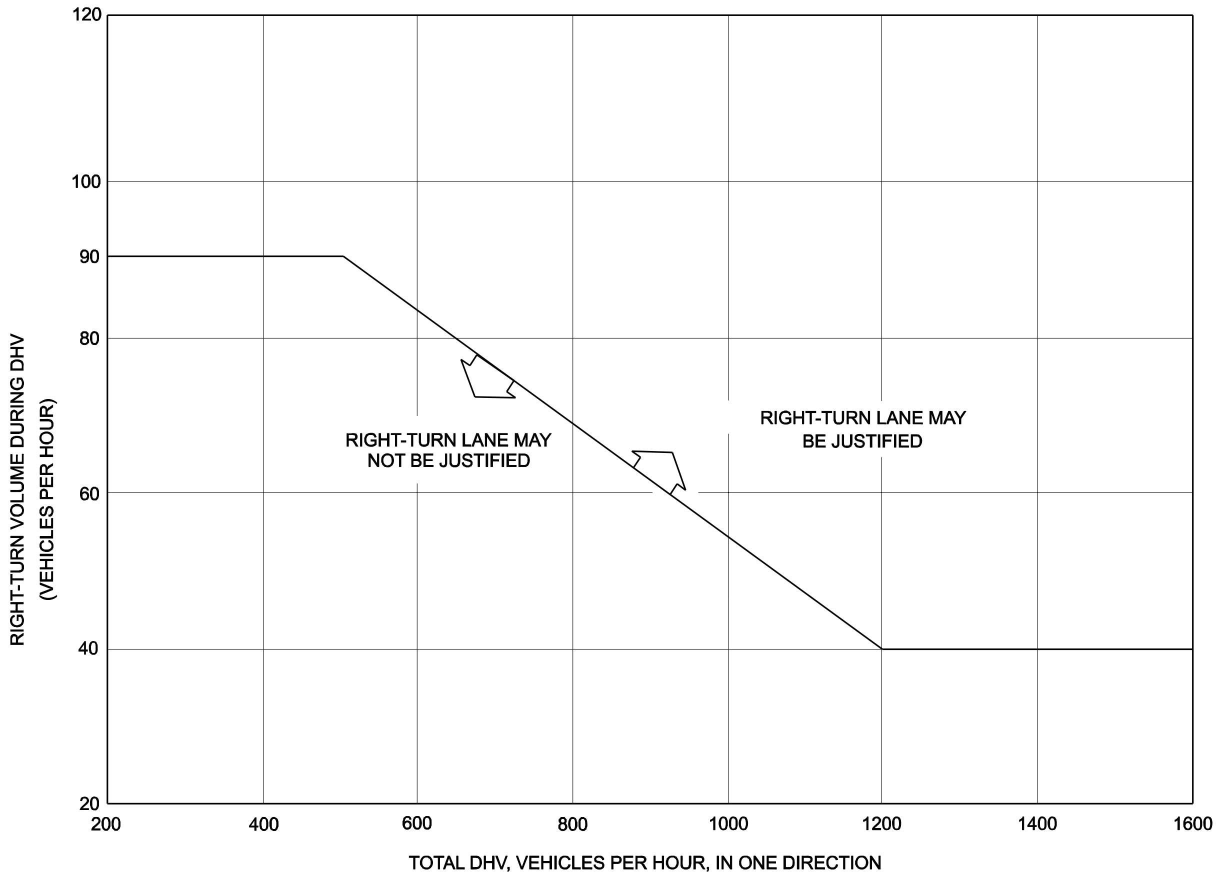
Given:	Design Speed	=	35 mph (60 km/h)
	DHV	=	250 vph
	Right Turns	=	100 vph

Problem: Determine if a right-turn lane is necessary.

Solution: To read the vertical axis, use 100 - 20 = 80 vph. The figure indicates that a right-turn lane is not necessary, unless other factors (e.g., high crash rate) indicate a lane is needed.

GUIDELINES FOR RIGHT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS

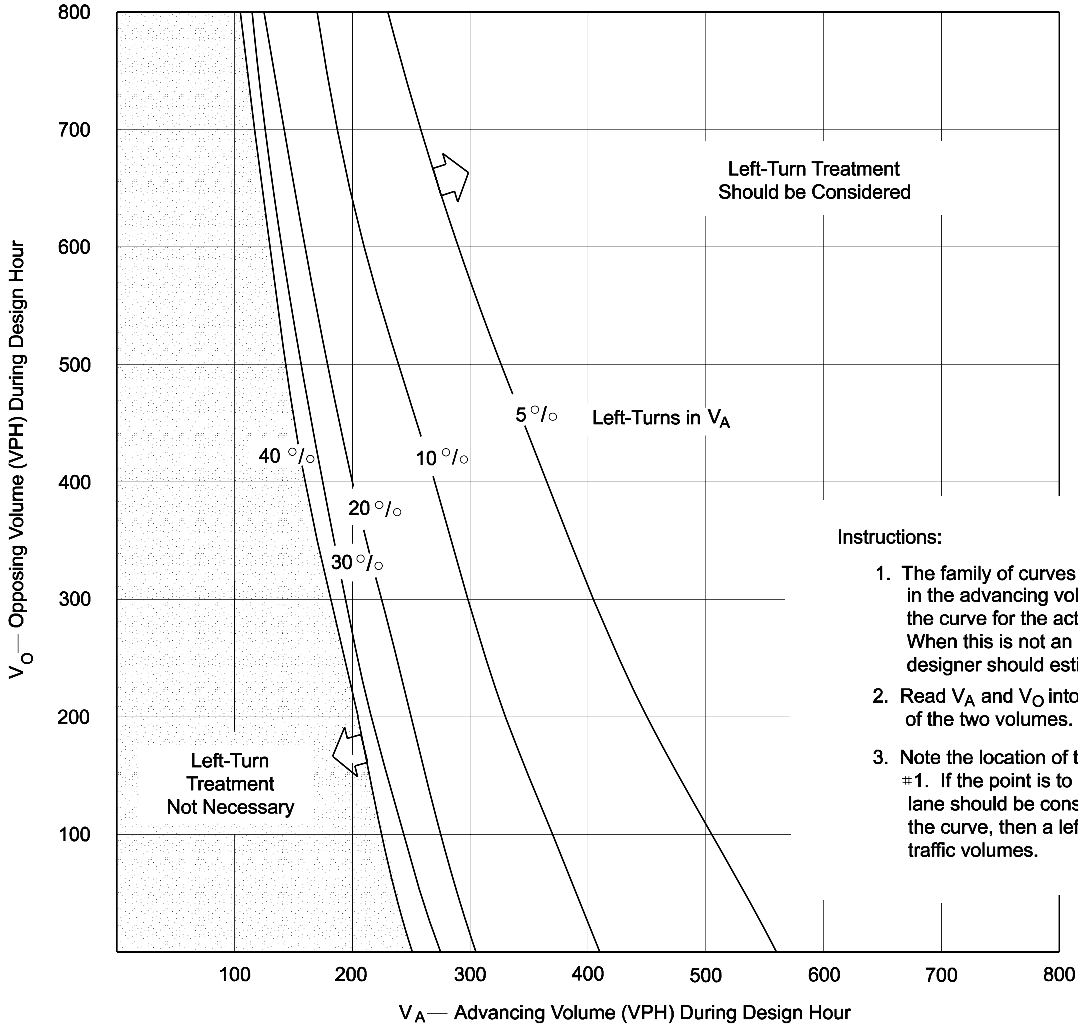
Figure 28.4A



Note: Figure is only applicable on highways with a design speed of 50 mph (80 km/h) or greater.

GUIDELINES FOR RIGHT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 4-LANE HIGHWAYS

Figure 28.4B



V_A = Total advancing traffic volume which includes all turning traffic

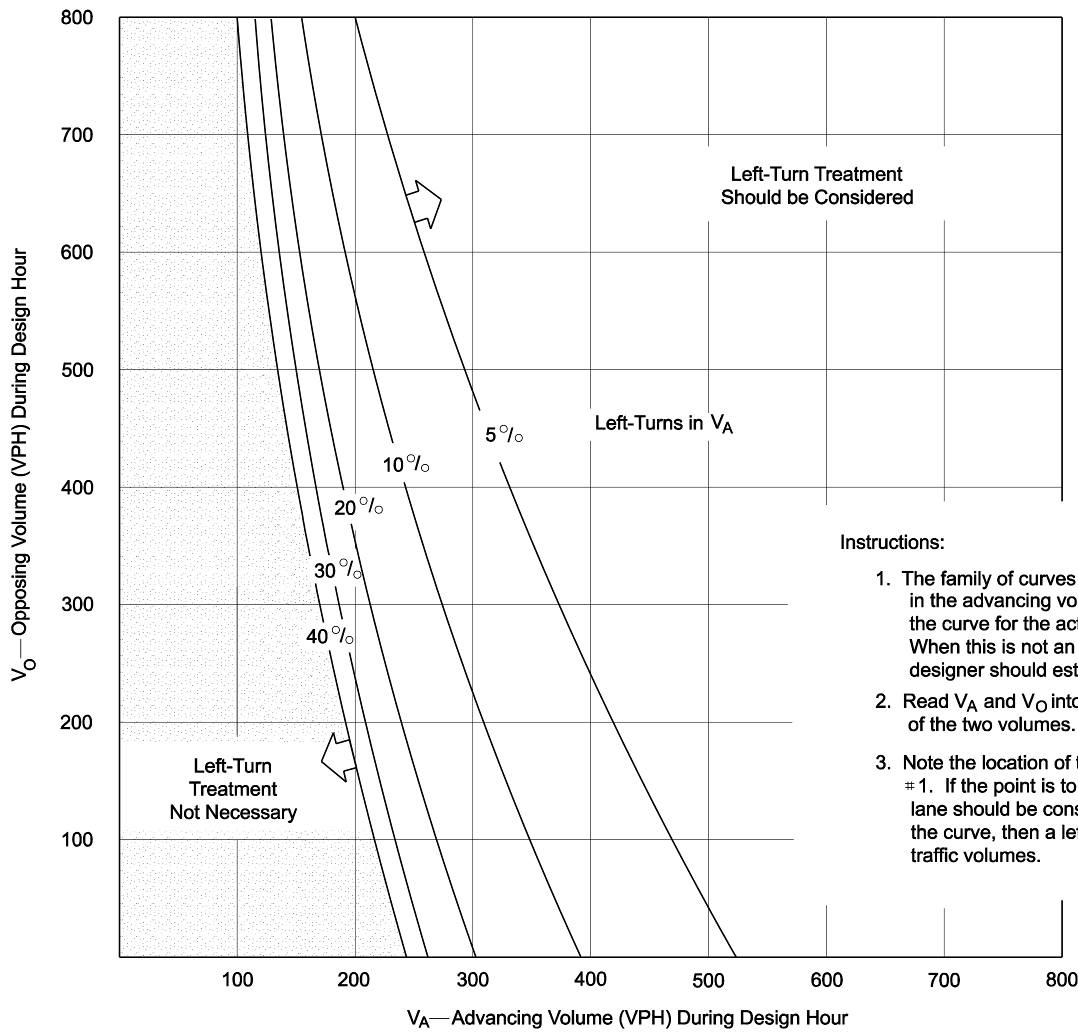
V_O = Total opposing traffic volume which includes all turning traffic

Instructions:

1. The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of five, the designer should estimate where the curve lies.
2. Read V_A and V_O into the chart and locate the intersection of the two volumes.
3. Note the location of the point in #2 relative to the curve in #1. If the point is to the right of the curve, then a left-turn lane should be considered. If the point is to the left of the curve, then a left-turn lane is not warranted based on traffic volumes.

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (60 MPH)
(US Customary)

Figure 28.4C



V_A = Total advancing traffic volume which includes all turning traffic

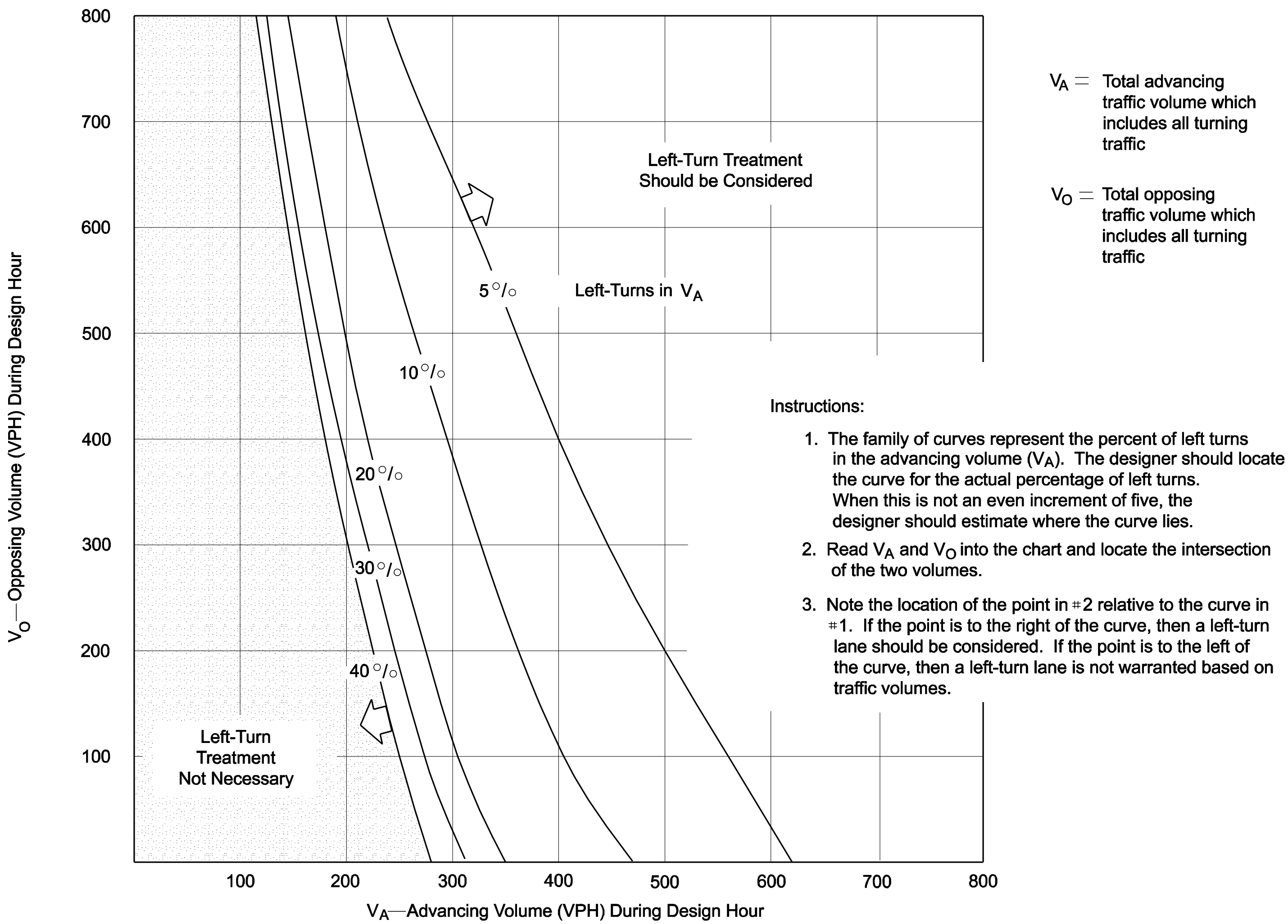
V_O = Total opposing traffic volume which includes all turning traffic

Instructions:

1. The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of five, the designer should estimate where the curve lies.
2. Read V_A and V_O into the chart and locate the intersection of the two volumes.
3. Note the location of the point in #2 relative to the curve in #1. If the point is to the right of the curve, then a left-turn lane should be considered. If the point is to the left of the curve, then a left-turn lane is not warranted based on traffic volumes.

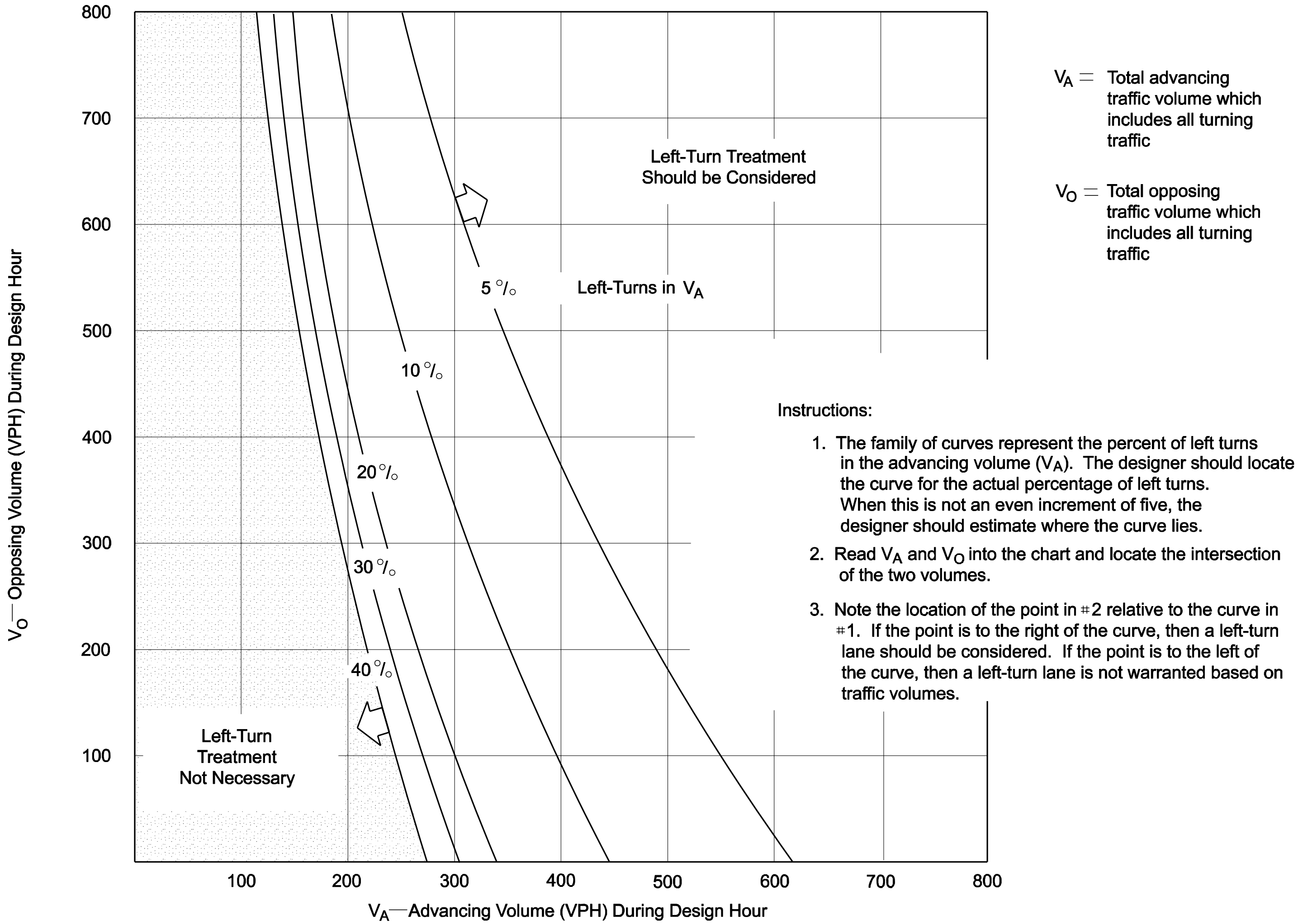
VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (100 KM/H) (Metric)

Figure 28.4C



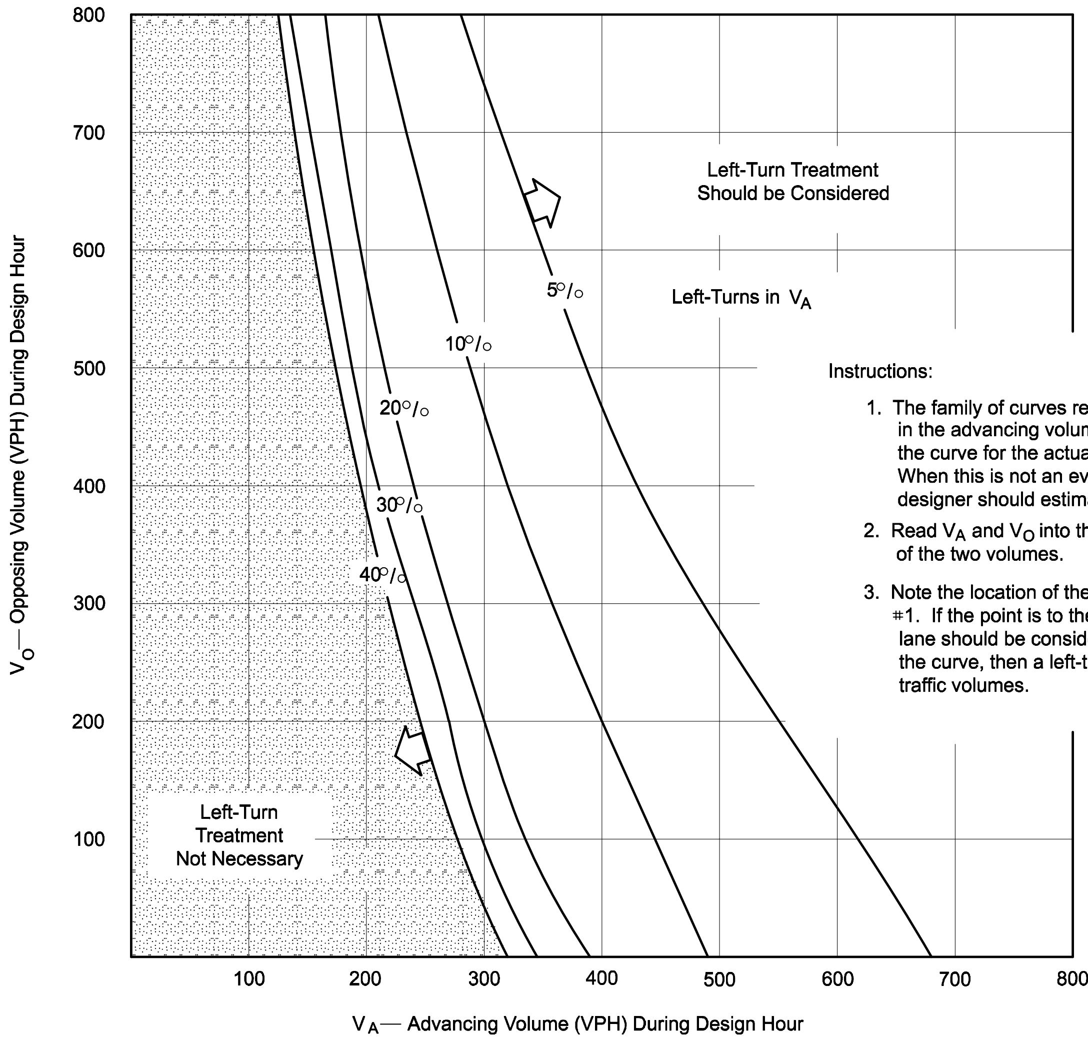
VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (55 MPH) (US Customary)

Figure 28.4D



VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (90 KM/H) (Metric)

Figure 28.4D



V_A = Total advancing traffic volume which includes all turning traffic

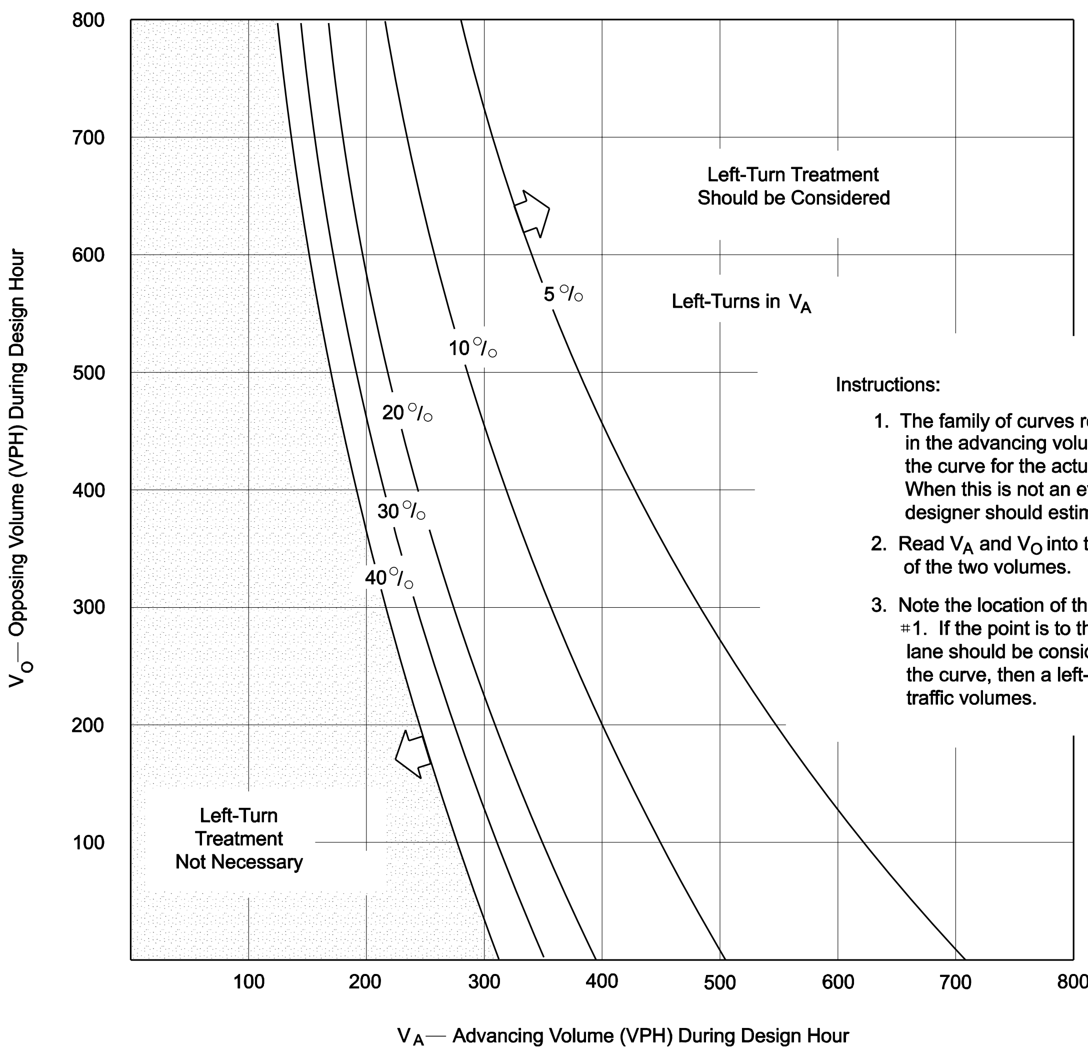
V_O = Total opposing traffic volume which includes all turning traffic

Instructions:

1. The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of five, the designer should estimate where the curve lies.
2. Read V_A and V_O into the chart and locate the intersection of the two volumes.
3. Note the location of the point in #2 relative to the curve in #1. If the point is to the right of the curve, then a left-turn lane should be considered. If the point is to the left of the curve, then a left-turn lane is not warranted based on traffic volumes.

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (50 MPH) (US Customary)

Figure 28.4E



V_A = Total advancing traffic volume which includes all turning traffic

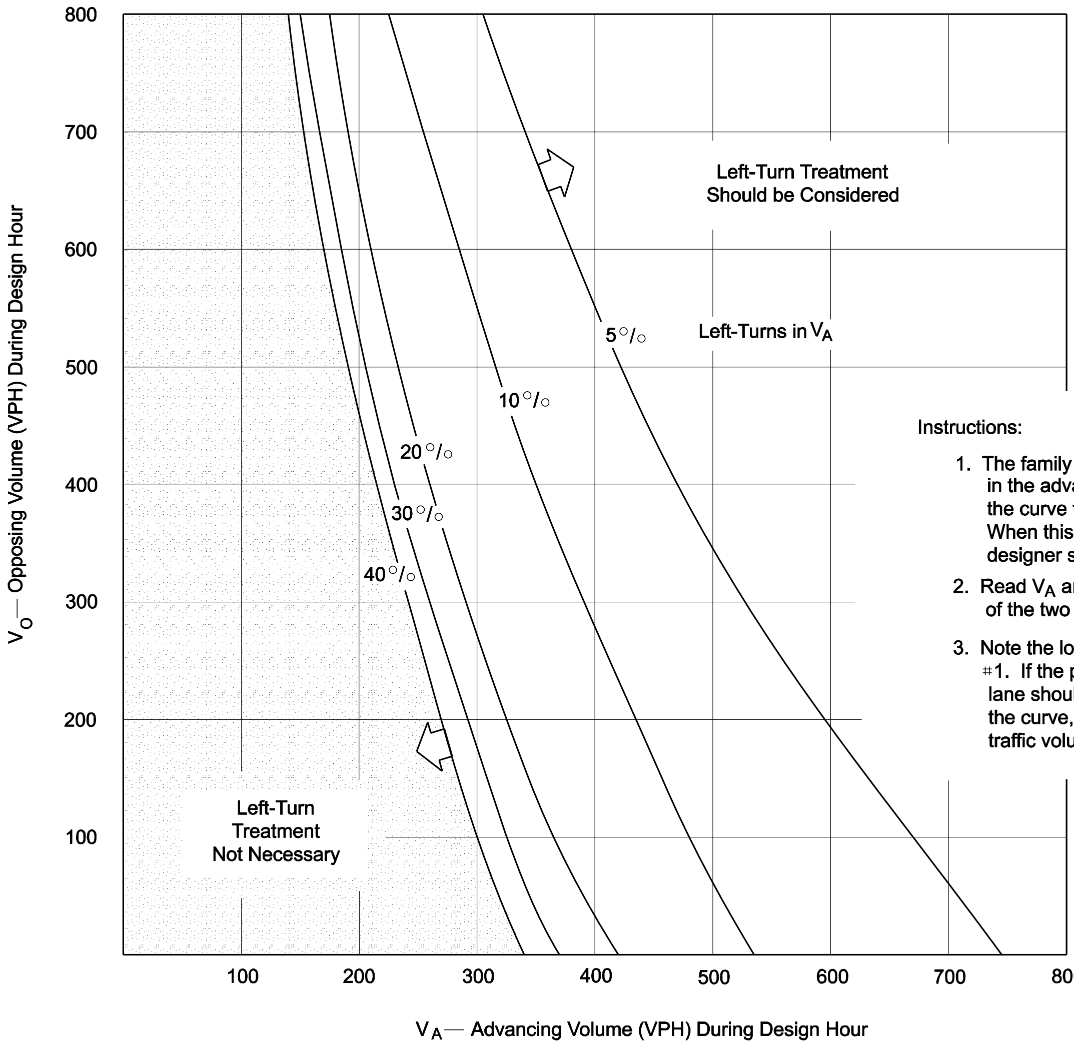
V_O = Total opposing traffic volume which includes all turning traffic

Instructions:

1. The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of five, the designer should estimate where the curve lies.
2. Read V_A and V_O into the chart and locate the intersection of the two volumes.
3. Note the location of the point in #2 relative to the curve in #1. If the point is to the right of the curve, then a left-turn lane should be considered. If the point is to the left of the curve, then a left-turn lane is not warranted based on traffic volumes.

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (80 KM/H) (Metric)

Figure 28.4E



V_A = Total advancing traffic volume which includes all turning traffic

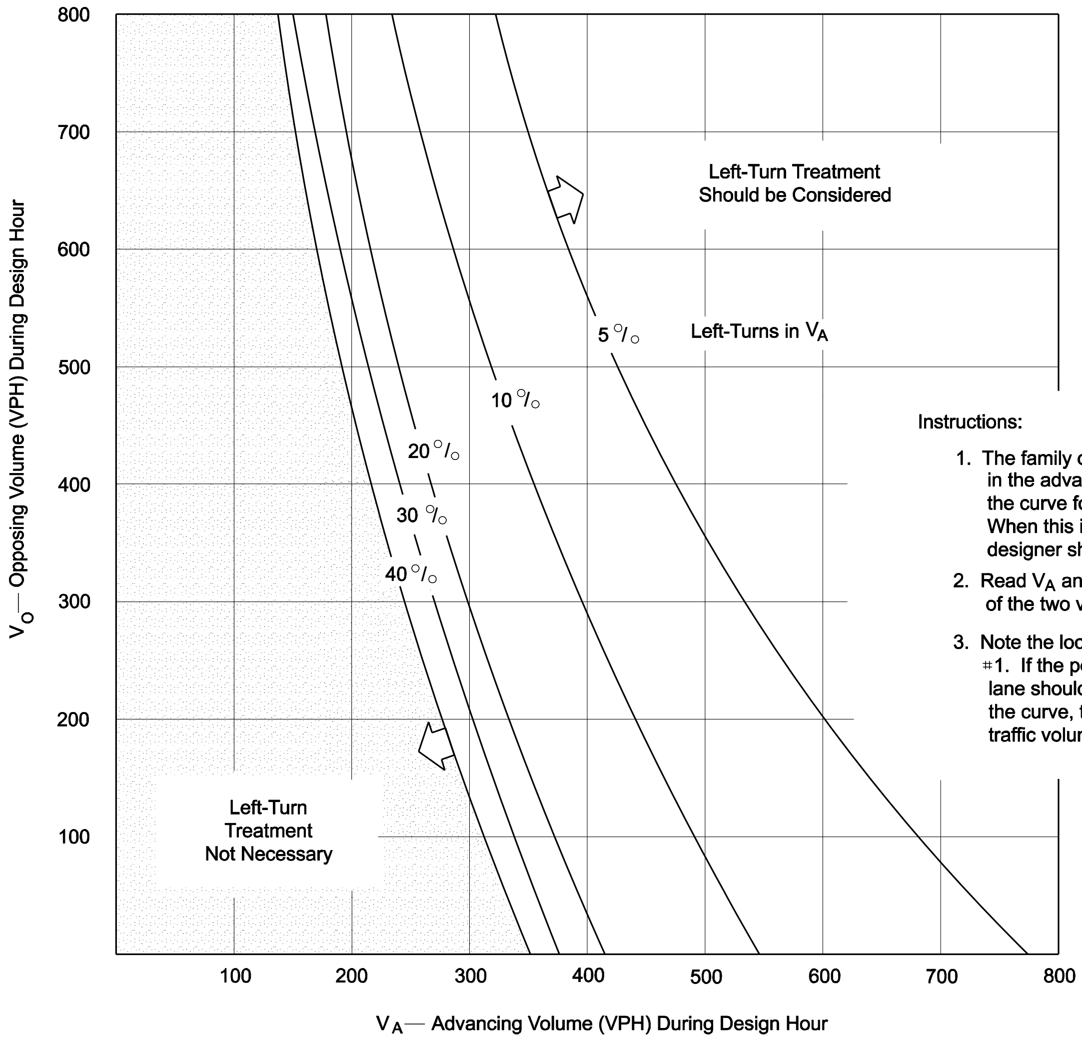
V_O = Total opposing traffic volume which includes all turning traffic

Instructions:

1. The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of five, the designer should estimate where the curve lies.
2. Read V_A and V_O into the chart and locate the intersection of the two volumes.
3. Note the location of the point in #2 relative to the curve in #1. If the point is to the right of the curve, then a left-turn lane should be considered. If the point is to the left of the curve, then a left-turn lane is not warranted based on traffic volumes.

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (45 MPH)
(US Customary)

Figure 28.4F



V_A = Total advancing traffic volume which includes all turning traffic

V_O = Total opposing traffic volume which includes all turning traffic

Instructions:

1. The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of five, the designer should estimate where the curve lies.
2. Read V_A and V_O into the chart and locate the intersection of the two volumes.
3. Note the location of the point in #2 relative to the curve in #1. If the point is to the right of the curve, then a left-turn lane should be considered. If the point is to the left of the curve, then a left-turn lane is not warranted based on traffic volumes.

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (70 KM/H) (Metric)

Figure 28.4F

28.4.2 Design of Turn Lanes

28.4.2.1 Widths

The following will apply to auxiliary turn lane widths:

1. Lane Widths. Typically, the width of any turn lanes at an intersection is the same as that of the adjacent through lane. In rare cases, it may be justified to provide a narrower width (e.g., restricted right-of-way).

2. Shoulder. The designer should meet the following for shoulders adjacent to auxiliary lanes:

   a. On uncurbed facilities, the shoulder width adjacent to the auxiliary lane should be the same as the normal shoulder width for the approaching roadway. At a minimum, the width may be 4 ft (1.2 m), assuming the roadway has a shoulder width equal to or greater than 4 ft (1.2 m).

   b. On curbed facilities, the offset between the auxiliary lane and face of curb should be the same as that for the normal roadway section, typically 2 ft (0.6 m). At a minimum, the offset may be 1 ft (0.3 m).

3. Cross Slope. The cross slope for an auxiliary lane will typically be the same as the adjacent through lane, which is typically 2%.

28.4.2.2 Turn Lane Lengths

The length of a right- or left-turn lane at an intersection should allow for both safe vehicular deceleration and storage of turning vehicles. This is the Department's minimum design at rural intersections. However, for urban facilities, it may be impractical to provide a turn lane length that provides for deceleration. Therefore, the full-width, turn-lane length may be designed to only provide sufficient distance for storage at urban intersections. To determine the turn lane length, the designer should consider the following:

1. Taper. For tapers, the following will apply:

   a. Design. A straight-line taper is typically used at the entrance of the turn lane.

   b. NHS Routes. The taper length is in addition to the deceleration distance as described in Comment #2 (i.e., the deceleration is assumed to begin after the taper).

c. Non-NHS Routes. The taper distance is included in the deceleration distance as described in Comment #2; (i.e., deceleration is assumed to begin at the beginning of the taper).

d. Taper Rates. Figure 28.4G provides the recommended taper rates for various design speeds.

2. Deceleration. For rural facilities, the deceleration distance (L_D) should meet the criteria presented in Figure 28.4H. This assumes that the driver will come to a complete stop before turning. For turning roadways, it can be assumed that the driver will be making the right turn at 15 mph (20 km/h). The deceleration distances for 15 mph (20 km/h) are also presented in Figure 28.4H. These distances are desirable on urban facilities; however, this is not always feasible. Under restricted urban conditions and where the design speed is less than or equal to 45 mph (70 km/h), deceleration may have to be accomplished entirely within the travel lane. For these cases, the length of turn lane will be determined solely on the basis of providing adequate vehicular storage (i.e., L_D = 0.0 ft (0.0 m)).

3. Storage. The storage length (L_S) for turn lanes should be sufficient to store the number of vehicles likely to accumulate. The designer should consider the following in determining the recommended storage length:

a. Signalized Intersections. Figure 28.4I illustrates the method to determine the recommended storage length for left-turn lanes, or right-turn lanes where right-turn-on-red is prohibited at a signalized intersection. The values obtained from the figure are for a cycle length of 75 seconds and a volume/capacity (*v/c*) ratio of 0.80. For other values, the designer should multiply the length obtained in the figure by an adjustment factor found in the accompanying table with Figure 28.4I. The *v/c* ratio is determined by a capacity analysis as described in the Highway Capacity Manual. The designer should also ensure at signalized intersections that the right- and left-turn lane lengths exceed the storage length of the adjacent through lane. Otherwise, a vehicular queue in the through lane will block entry into the turn lane for turning vehicles. Most capacity software packages have a queuing model available. Use engineering judgment to determine the appropriate method to use to determine storage requirements.

b. Unsignalized Intersections. The minimum storage length should be sufficient to accommodate the expected number of turning vehicles likely to arrive in an average 2 minute period within the design hour. The

US Customary	Metric	Taper Rate
Design Speed (mph)	Design Speed (km/h)	
$V < 35$	$V < 60$	8:1
$35 \leq V < 50$	$60 \leq V < 80$	10:1
$50 \leq V < 55$	$80 \leq V < 90$	15:1
$55 \leq V$	$90 \leq V$	18:1

RECOMMENDED BAY TAPER RATES

Figure 28.4G

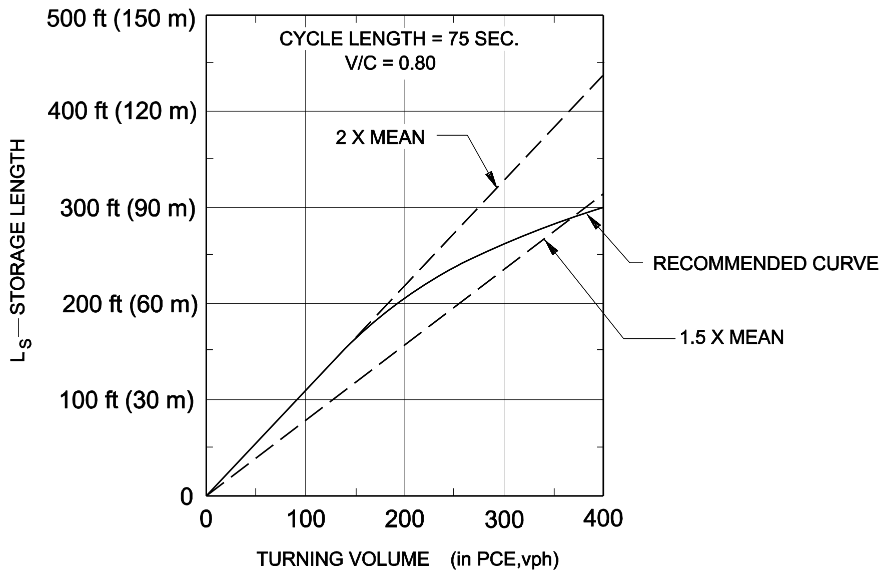
US Customary			
Design Speed (mph)	Average Running Speed(1) (mph)	L_D (ft)	
		Stop Condition	15 mph
70	58	615	590
65	55	570	540
60	52	530	500
55	48	480	455
50	44	435	405
45	40	385	350
40	36	320	295
35	32	280	250
30	28	235	200
25	25	200	185
Metric			
Design Speed (km/h)	Average Running Speed(1) (km/h)	L_D (m)	
		Stop Condition	20 km/h
110	91	180	180
100	85	170	165
90	77	145	140
80	70	130	125
70	63	110	105
60	55	95	90
50	47	75	70
40	40	60	55

(1) Average running speeds assumed for calculations.

Note: Bay taper may be included in the deceleration length on non-NHS projects.

DECELERATION DISTANCES FOR TURN LANES

Figure 28.4H



Storage Length Adjustment Factors

v/c RATIO, *X*	CYCLE LENGTH, *C* (SEC)				
	60	70	80	90	100
0.50	0.70	0.76	0.84	0.89	0.94
0.55	0.71	0.77	0.85	0.90	0.95
0.60	0.73	0.79	0.87	0.92	0.97
0.65	0.75	0.81	0.89	0.94	1.00
0.70	0.77	0.84	0.92	0.98	1.03
0.75	0.82	0.88	0.98	1.03	1.09
0.80	0.88	0.95	1.05	1.11	1.17
0.85	0.99	1.06	1.18	1.24	1.31
0.90	1.17	1.26	1.40	1.48	1.56
0.95	1.61	1.74	1.92	2.03	2.14

Notes:

1. *Figure applies to exclusive left-turn lanes and exclusive right-turn lanes where right-turns-on-red are not allowed.*
2. *See minimum storage length discussion in Section 28.4.2.2.*
3. *To determine the v/c ratio and the passenger car equivalent (PCE) values, see the Highway Capacity Manual.*
4. *If turning volumes exceed 300 vph, consider providing dual-turn lanes.*

RECOMMENDED STORAGE LENGTH FOR SIGNALIZED INTERSECTIONS

Figure 28.4I

recommended storage lengths for right- and left-turn lanes at unsignalized intersections are provided in Figure 28.4J. Under restricted conditions, the following formula may be used:

$$L_s = \frac{DHV}{30} \text{ x Vehicle Spacing} \qquad \text{(Equation 28.4-1)}$$

Where:

L_S	=	turn lane storage length, ft (m)
DHV	=	the number of vehicles turning during the design hour
Vehicle Spacing	=	the distance between the fronts of two stopped vehicles. For passenger cars, this distance is assumed to be 25 ft (7.5 m).

c. Minimum Turn-Lane Length. In urban areas, the minimum full-width length for storage is 50 ft (15 m) where there are less than 10% trucks and 100 ft (30 m) where there are 10% or more trucks. For rural areas, the minimum storage length may be 0 ft (0.0 m).

Turning DHV	US Customary	Metric
	L_S (ft)	L_S (m)
≤ 60	0 (rural); 50 – 75 (urban)	0 (rural); 15-25 (urban)
61 – 120	100	30
121 – 180	150	45
> 180	200 or greater	60 or greater

Note: See Section 28.4.2.2 for minimum storage length criteria.

RECOMMENDED STORAGE LENGTHS (L_S) FOR UNSIGNALIZED INTERSECTIONS

Figure 28.4J

28.4.3 Typical Turn Lane Treatments

Various turn lane designs are illustrated in Figures 28.4K through 28.4N. In addition, the designer should consider the following:

1. Two-Lane Facilities. If a left-turn lane is required on a 2-lane highway, it should desirably be designed as a fully channelized left-turn lane. Typical channelized left-turn lanes on a NHS facility are illustrated in Figures 28.4K and 28.4L. Note, for a turn lane on a non-NHS facility, the taper is included in the deceleration length. Generally, left-turn deceleration and storage bays will be designed symmetrically about the highway centerline for both flush and raised medians.

2. Divided Facilities. Figure 28.4M illustrates typical treatments for left- and right-turn lanes on divided NHS facilities. Left-turn lanes will generally be the parallel design. To properly develop the left-turn lane, the median should be at least 16 ft (4.8 m) wide, 12 ft (3.6 m) for the turn lane and 4 ft (1.2 m) for the raised median.

3. Offset Left-Turn Lanes. On medians wider than 18 ft (5.4 m), it is desirable to align the left-turn lane so that it will reduce the width of the median "nose" to 1 ft (0.3 m) (painted) or to 4 ft (1.2 m) (raised). This alignment will place the vehicle waiting to make the turn as far to the left as practical, maximize the offset between the opposing left-turn lanes and provide improved visibility to the opposing through traffic. The advantages of offsetting the left-turn lanes are:

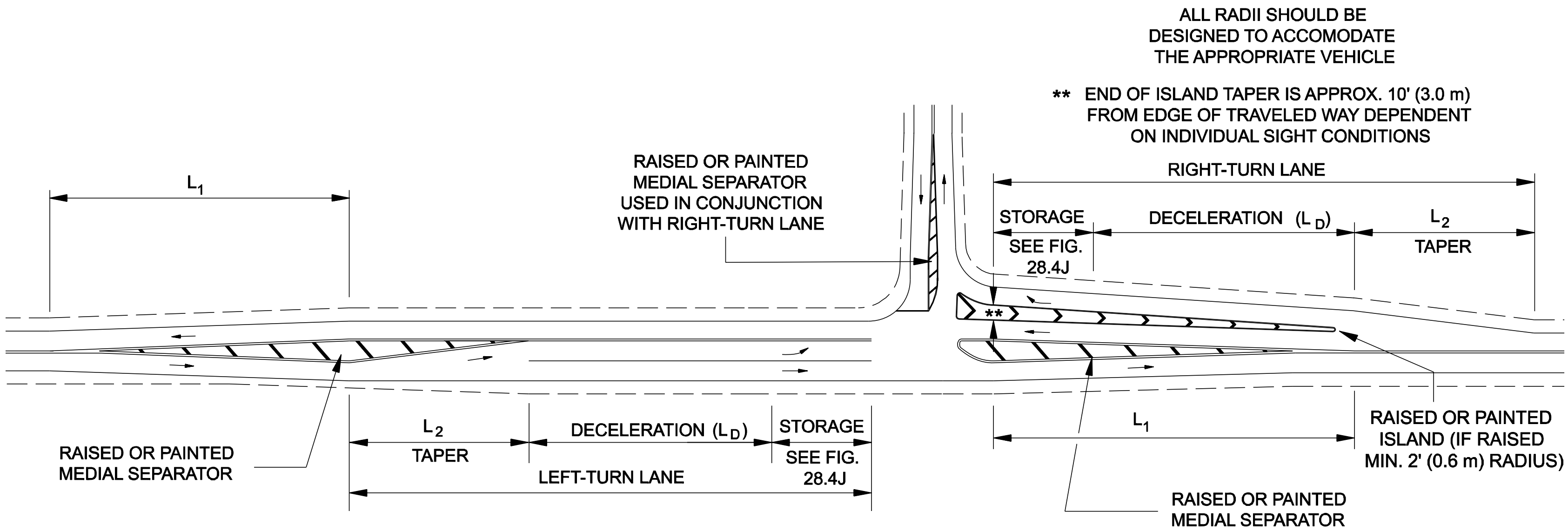
    a. better visibility of opposing through traffic;

    b. decreased probability of a conflict between opposing left-turn movements within the intersection; and

    c. more left-turn vehicles can be served in a given period of time, especially at signalized intersections.

    Offset designs may be either the parallel or taper design; see Figure 28.4N. The parallel design may be used at signalized and unsignalized intersections. However, the taper design is primarily only used at signalized intersections. Offset turn lanes should be separated from the adjacent through traveled way by painted or raised channelization.

4. Right-Turn Deceleration Lanes. Deceleration vehicles in right-turn lanes can cause sight obstructions for vehicles on the side approach. High speed through traffic can be shadowed in the sight triangle for traffic on the side approach. Consider this visibility concern in the design of right-turn lanes. This may require additional lateral shift of the turn lane to provide positive separation between the



Design Speed (mph)	Taper Rate	
	Lane Shifts (L_1)	Auxiliary Lanes (L_2)
20	10:1	8:1
25	15:1	8:1
30	20:1	8:1
35	25:1	10:1
40	40:1	10:1
45	45:1	10:1
50	50:1	15:1
55	55:1	18:1
60	60:1	18:1
65	65:1	18:1
70	70:1	18:1
75	75:1	18:1

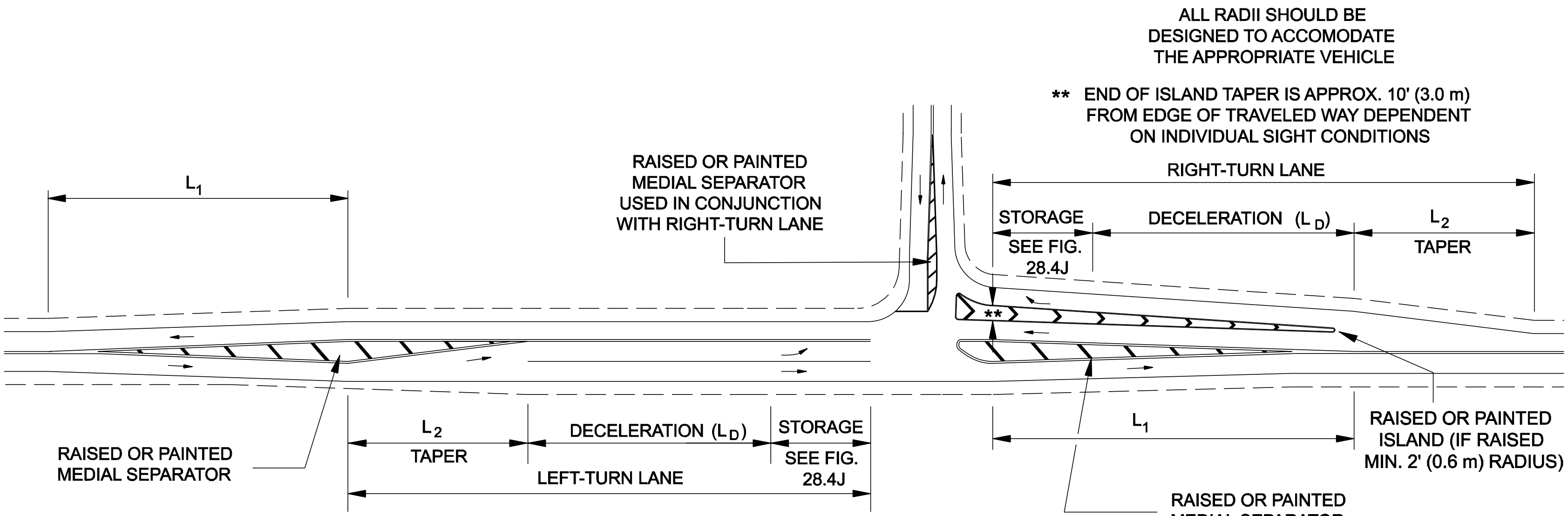
Taper Length (L) = Taper Rate x Offset Distance
See Section 28.4.2.2 for minimum left-turn lane lengths.

Design Speed (mph)	Average Running(1) Speed (mph)	L_D(ft)(2)	
		Stop Condition	15 mph
25	25	200	185
30	28	235	200
35	32	280	250
40	36	320	295
45	40	385	350
50	45	435	405
55	48	480	455
60	52	530	500
65	55	570	540
70	58	615	590

(1) Average running speeds assumed for calculations.
(2) Bay taper may be included in the deceleration length on non-NHS projects.

CHANNELIZED TURN LANES FOR 2-LANE FACILITIES
(US Customary)

Figure 28.4K



Design Speed (km/h)	Taper Rate	
	Lane Shifts (L_1)	Auxiliary Lanes (L_2)
30	10:1	8:1
40	15:1	8:1
50	20:1	8:1
60	25:1	10:1
70	45:1	10:1
80	50:1	15:1
90	55:1	18:1
100	60:1	18:1
110	70:1	18:1
120	75:1	18:1

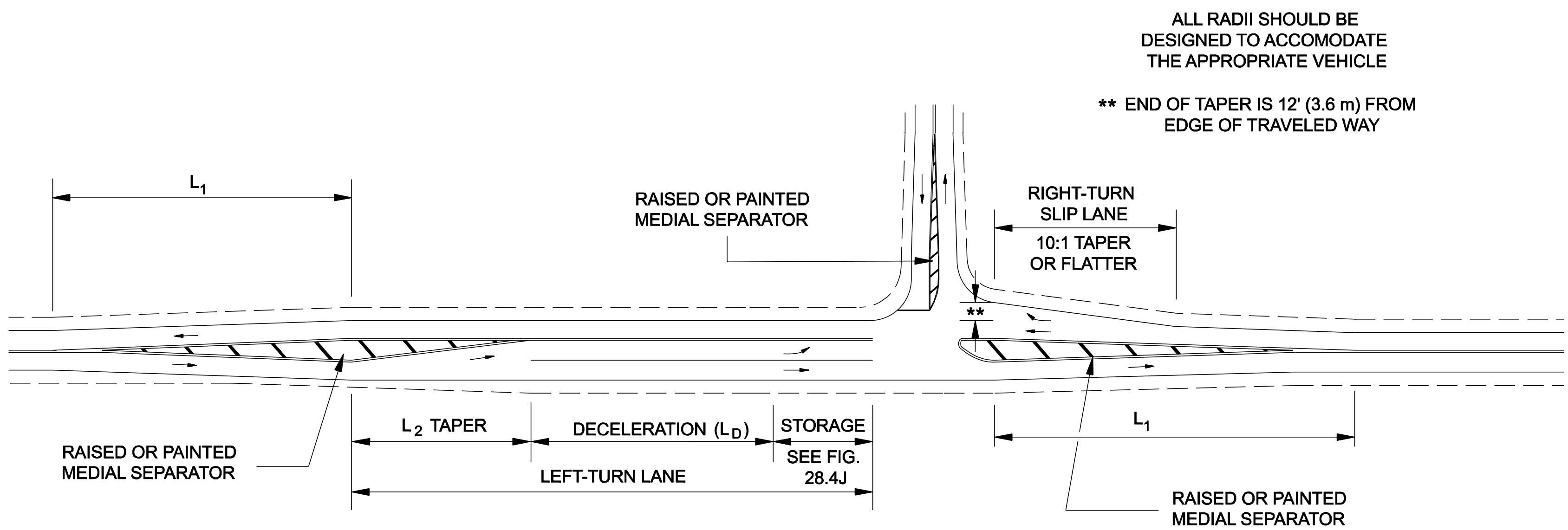
Taper Length (L) = Taper Rate x Offset Distance
See Section 28.4.2.2 for minimum left-turn lane lengths.

Design Speed (km/h)	Average Running(1) Speed (km/h)	L_D(m)(2)	
		Stop Condition	20 km/h
40	40	60	55
50	47	75	70
60	55	95	90
70	63	110	105
80	70	130	125
90	77	145	140
100	85	170	165
110	91	180	180

(1) Average running speeds assumed for calculations.
(2) Bay taper may be included in the deceleration length on non-NHS projects.

CHANNELIZED TURN LANES FOR 2-LANE FACILITIES
(Metric)

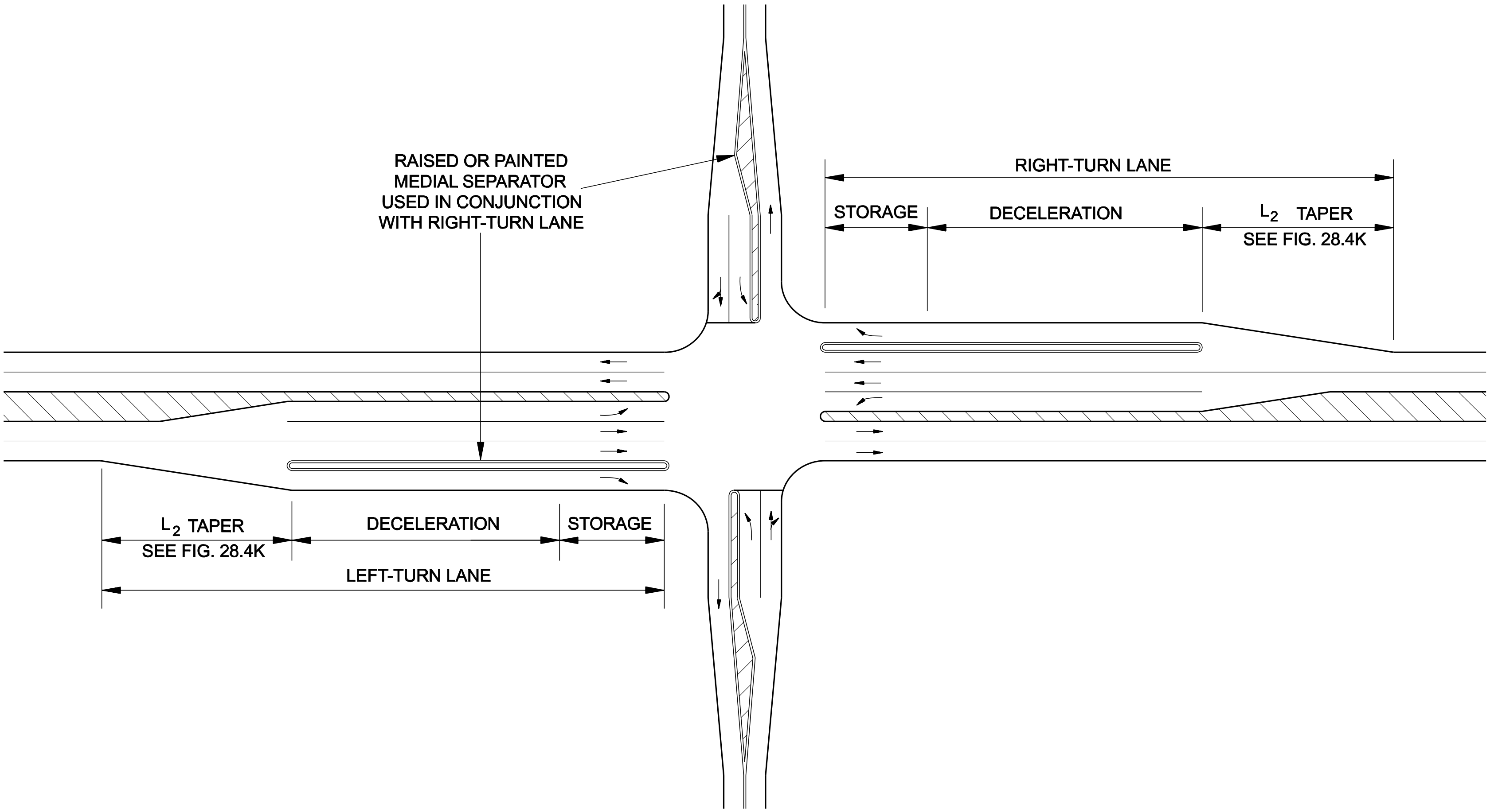
Figure 28.4K



Note: See Figure 28.4K for L_1, L_2 and L_D distances.

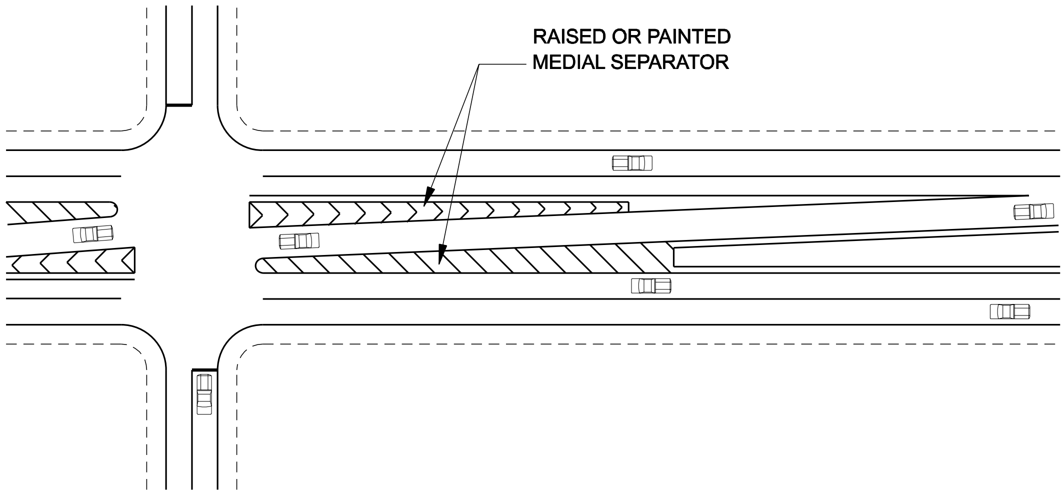
CHANNELIZED LEFT-TURN LANE FOR 2-LANE FACILITIES

Figure 28.4L

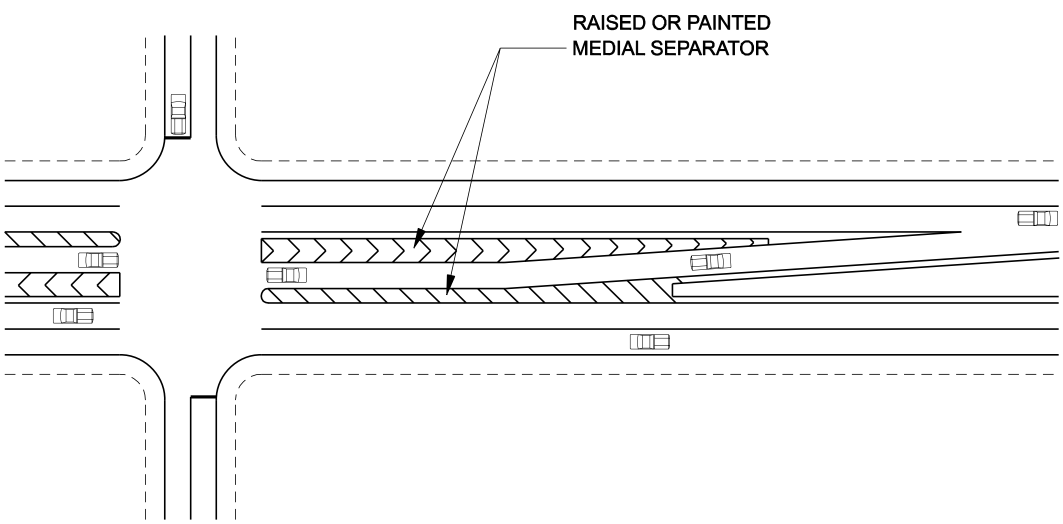


CHANNELIZED TURN LANES FOR 4-LANE FACILITIES

Figure 28.4M



a) TAPERED-OFFSET TURN LANE



b) PARALLEL-OFFSET TURN LANE

OFFSET LEFT-TURN LANES

Figure 28.4N

right-turn lane and the through lane. It is also imperative that the design of the side approach keeps the stop bar as close to the intersection as practical. This may require a medial separation and/or channelizing islands on the side approach to give positive guidance to the proper position of a stopped vehicle.

28.4.4 Dual Turn Lanes

28.4.4.1 Guidelines

At signalized intersections with high-turning volumes, dual left- and/or right-turn lanes may be considered. However, multiple turn lanes may cause problems with right-of-way, lane alignment, crossing pedestrians and lane confusion for approaching drivers. Consider dual right- and left-turn lanes where:

1. based on the capacity analysis, the necessary time for a protected left-turn phase becomes unattainable to meet the level-of-service criteria (average delay per vehicle);

2. there is insufficient space to provide the calculated length of a single-turn lane because of site restrictions (e.g., closely spaced intersections); and/or

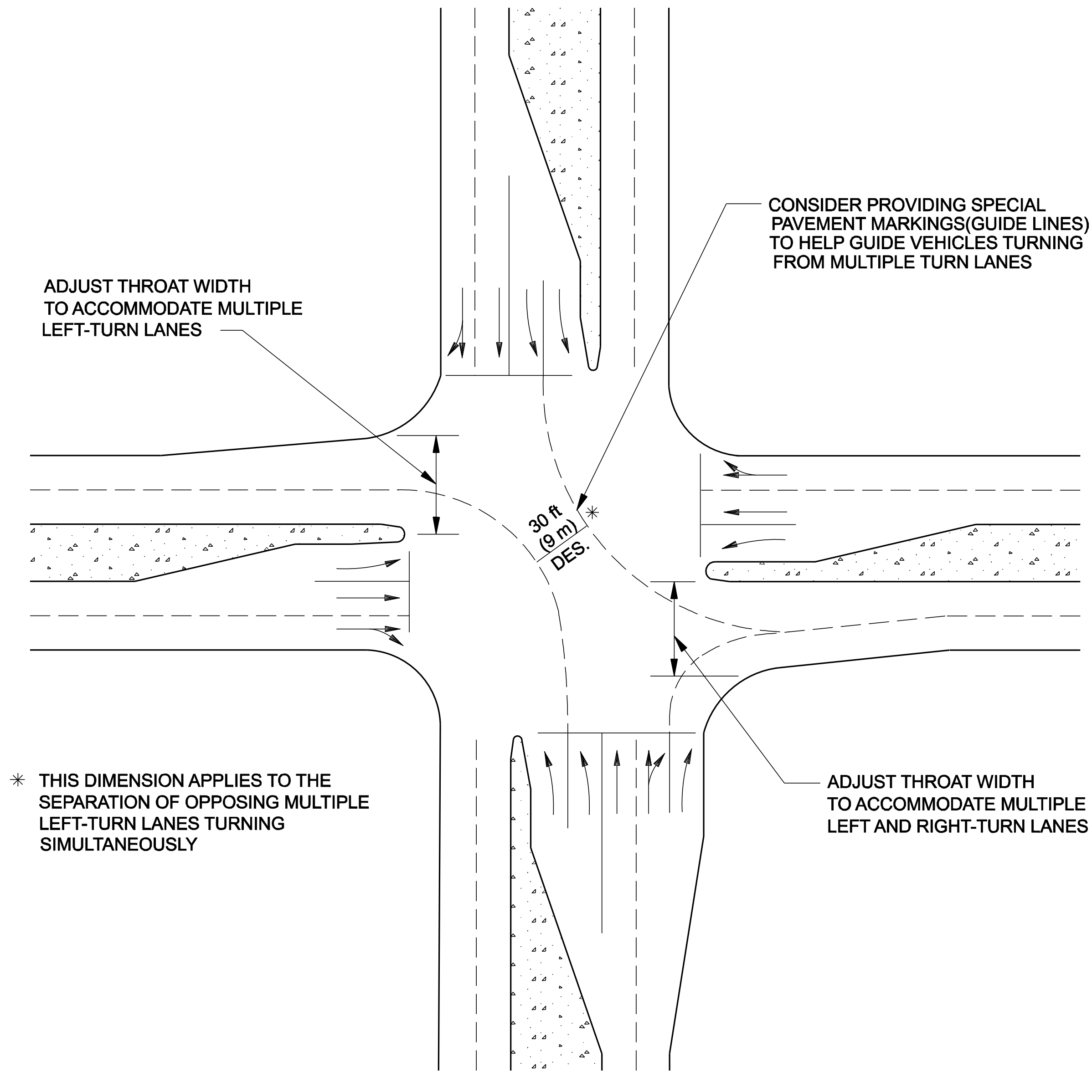
3. the calculated length of a single-turn lane becomes prohibitive.

Dual right-turn lanes do not work as well as dual left-turn lanes because of the more restrictive space available for two-abreast right turns. If practical, the designer should find an alternative means to accommodate the high number of right-turning vehicles. For example, a turning roadway may accomplish this purpose.

28.4.4.2 Design

A dual-turn lane (both lanes exclusive) can potentially discharge approximately 1.9 times the number of cars that will discharge from a single exclusive turn lane. Figure 28.4O presents both dual right- and left-turn lanes to illustrate the more important design elements. Several design elements that must be carefully considered are included in the following:

1. Throat Width. Because of the off-tracking characteristics of turning vehicles, the normal width of two travel lanes may be inadequate to properly receive two vehicles turning abreast. Therefore, the receiving throat width may need to be widened. For 90° intersections, the designer can expect that the throat width for dual turn lanes will be approximately 30 ft to 36 ft (9 m to 11 m). If the angle of



DUAL TURN LANES

Figure 28.4O

turn is less than 90°, it may be acceptable to provide a narrower width. When determining the available throat width, the designer can assume that the paved shoulder, if present, will be used to accommodate two-abreast turns.

2. Widening Approaching Through Lanes. If a 30 ft to 36 ft (9 m or 11 m) throat width is provided to receive dual-turn lanes, the designer should also consider how this will affect the traffic approaching from the other side. The designer should also ensure that the through lanes line up relatively well to ensure a smooth flow of traffic through the intersection.

3. Pavement Markings. Section 19.4.6 illustrates and discusses the applicable pavement markings that are necessary to effectively guide the two lines of vehicles turning abreast.

4. Opposing Left-Turn Traffic. If simultaneous, opposing left turns will be allowed, the designer should ensure that there is sufficient space for all turning movements. This is always a factor, but dual left-turn lanes can cause special problems. If space is unavailable, it may be necessary to alter the signal phasing to allow the two directions of traffic to move through the intersection on separate phases.

5. Turning Templates. All intersection design elements for dual turn lanes must be checked by using the applicable turning templates or computer simulated turning template. The designer should assume that the selected design vehicle will turn from the inside turn lane. Desirably, the outside design vehicle should be an SU but, as a minimum, the other vehicle can be assumed to be a passenger vehicle turning side-by-side with the selected design vehicle.

28.5 TURNING ROADWAYS

Turning roadways connect intersecting roadways typically at non-signalized, at-grade intersections and allow a moderate speed, free-flowing right turn. Interchange ramps are not considered turning roadways.

28.5.1 Guidelines

Turning roadways should only be used when a thorough evaluation of the intersection has been completed. Consideration may be given to turning roadways where:

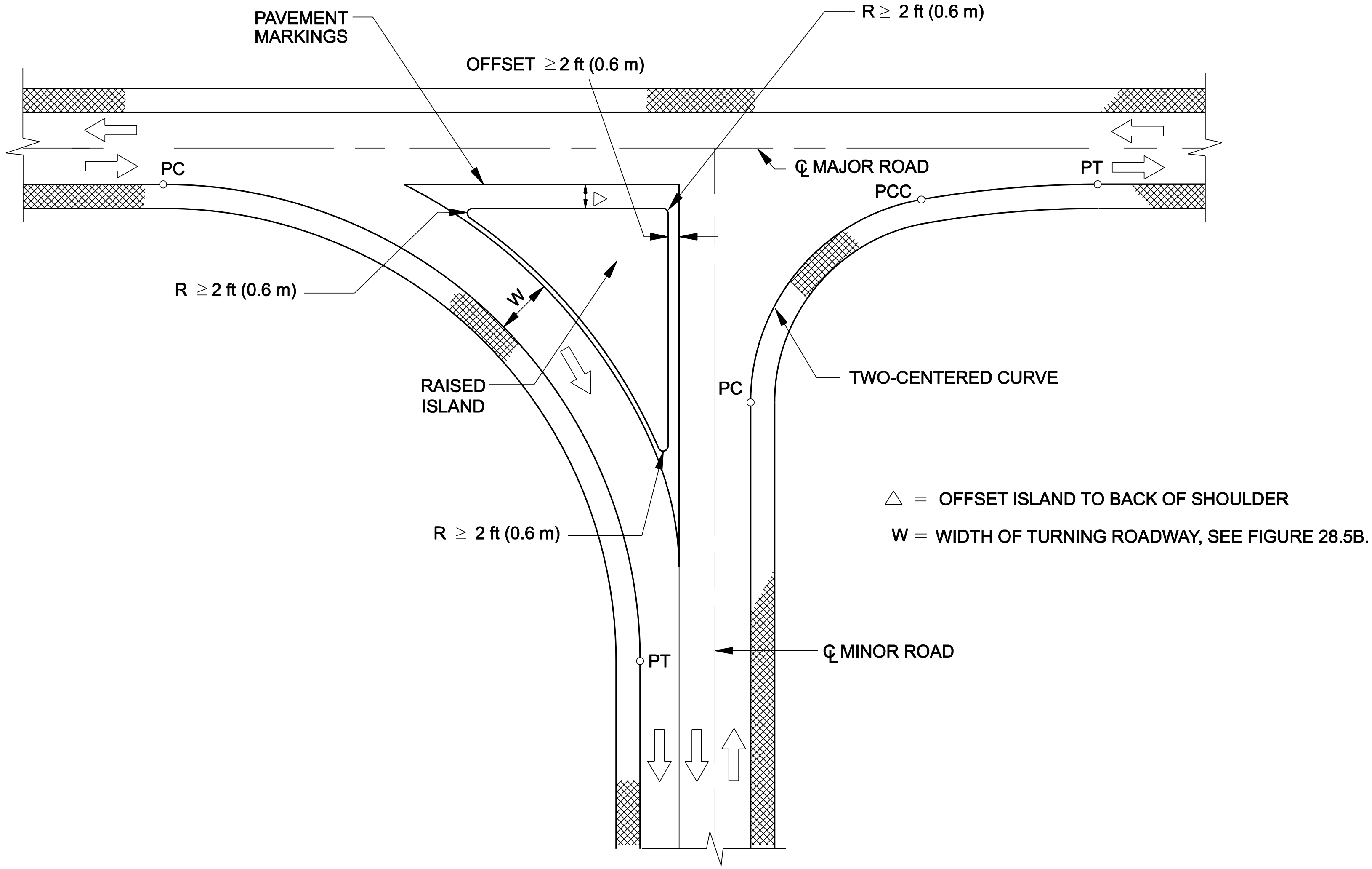
1. the design of a right-turn lane inhibits intersection sight distance;
2. it is desirable to allow right turns at 15 mph (20 km/h) or more on, for example, rural or urban arterials;
3. pedestrian crossing activity is low to non-existent;
4. the turning traffic flow is large in volume and the intersecting roadways are of similar functional class (e.g., arterial to arterial); and/or
5. the conflicts with approaches are minimal.

28.5.2 Design Criteria

Figure 28.5A illustrates a typical design for a turning roadway. The following sections provide additional guidance on the design of a turning roadway.

28.5.2.1 Design Speed

The design speed of the turning roadway is controlled by the design speeds of the departing and entering roadways.



TYPICAL TURNING ROADWAY DESIGN

Figure 28.5A

28.5.2.2 Width

Turning roadway widths are dependent upon the turning radii and design vehicle selected. Figure 28.3B provides the criteria for selection of the appropriate design vehicle. Figure 28.5B presents the turning roadway pavement widths for various design vehicles based on a 1-lane, one-way operation with no provision for passing a stalled vehicle. This design is generally appropriate for most at-grade intersections. The pavement widths in Figure 28.5B provide an extra 6 ft (1.8 m) clearance beyond the design vehicle's swept path. This additional width provides extra room for maneuverability, driver variances and the occasional larger vehicle.

28.5.2.3 Radii Designs

For radii design templates, see the AASHTO A Policy on Geometric Design of Highways and Streets.

28.5.2.4 Acceleration/Deceleration Lanes

Acceleration/deceleration lanes are especially beneficial where mainline and turning volumes are high. At these intersections, the deceleration lane may also be needed for storage. An acceleration lane for the exiting portion of the turning roadway may also be justified; however, it may not be beneficial if the turning roadway will be stop controlled. Consider using acceleration and deceleration lanes at intersections that include turning roadways for arterials with a design speed of 50 mph (80 km/h) or more. See Section 28.4 for the design details of the auxiliary lanes.

US Customary					
Radius on Inner Edge of Pavement, R(ft)	Width of Turning Roadways (ft)				
	P	City-BUS	SU	WB-50	WB-67
50	13	21	18	32	49
75	13	19	17	25	32
100	13	18	16	22	27
150	12	17	15	19	22
200	12	16	15	18	20
300	12	16	15	17	18
400	12	16	15	17	18
500	12	16	15	17	18
Tangent	12	15	14	15	15

Metric					
Radius on Inner Edge of Pavement, R(m)	Width of Turning Roadways (m)				
	P	City-BUS	SU	WB-15	WB-20
15	4.0	6.5	5.5	9.7	15.7
25	3.9	5.6	5.0	7.2	9.0
30	3.8	5.4	4.9	6.7	8.1
50	3.7	5.0	4.6	5.7	6.5
75	3.7	4.8	R4.5	5.3	5.9
100	3.7	4.8	4.5	5.3	5.9
125	3.7	4.8	4.5	5.3	5.9
150	3.7	4.8	4.5	5.3	5.9
Tangent	3.6	4.4	4.2	4.4	4.4

Notes:

1. *If vertical curb is used on one side, then a curb offset of 1 ft (0.3 m) should be added to the table value.*
2. *If vertical curb is used on both sides, then a curb offset of 2 ft (0.6 m) (1 ft (0.3 m) on each side) should be added to the table value.*
3. *Only use the turning roadways in this figure as a guide and check the final design with the applicable turning template or computer simulated turning template program*

PAVEMENT WIDTHS FOR TURNING ROADWAYS

Figure 28.5B

28.6 INTERSECTION ACCELERATION LANES

It may be advantageous to provide an acceleration lane for turning vehicles at high-volume, non-signalized intersections to allow turning vehicles to accelerate before merging with the through traffic. Note on low-volume streets, motorists generally will not use acceleration lanes.

28.6.1 Guidelines for Acceleration Lanes for Right-Turning Vehicles

The following provides general guidelines for the consideration of an acceleration lane for right-turning vehicles:

1. where there is sufficient right-of-way available to place the acceleration lane without passing through downstream intersections and/or private approaches (access should be controlled throughout the acceleration lane and an appropriate distance downstream);
2. where the intersection is near or at capacity (LOS E) in the design year;
3. where a turning roadway is used (see Section 28.5);
4. where the turning traffic at an unsignalized intersection must merge with a high-speed, high-volume facility;
5. where there is a significant history of rear-end and/or sideswipe crashes associated with the right-turn movement downstream of the intersection;
6. where there is inadequate intersection sight distance available; and/or
7. where there are high volumes of trucks turning onto the mainline.

Do not provide acceleration lanes at signalized intersections.

28.6.2 Design Criteria

Consider the following criteria when designing an acceleration lane at an intersection:

1. Types. Acceleration lanes at intersections are typically the parallel design. Chapter Twenty-nine provides additional information on acceleration lanes for interchange ramps which is also applicable to acceleration lanes at intersections.
2. Lengths. Right-turn acceleration lanes should meet the criteria presented in Chapter Twenty-nine. The “controlling curve” at an intersection is the design

speed of the turning roadway or the speed at which a vehicle can make the right or left turn, usually less than 15 mph (20 km/h). The designer may consider lengthening the acceleration lane:

a. where there is significant number of turning trucks;

b. where a longer distance is required to achieve the merge speed (e.g., steep upgrades); and/or

c. where traffic volumes on the mainline are significant and additional distance is required to find acceptable gaps in the traffic flow.

3. Taper. See Chapter Twenty-nine for the applicable taper rates.

28.7 CHANNELIZATION

28.7.1 Functional Types

Channelization can be grouped into the following functional types:

1. Medial Separators. Medial separators segregate opposing traffic flows. These separators are often introduced on side streets to channelize the turn paths of vehicles entering the side street. This prevents conflicts with stored vehicles stopped at the stop bar and improves the intersection sight triangles.

2. Islands. Islands at or near crosswalks channelize right-turning vehicles and aid or protect pedestrians crossing a wide roadway. These islands may be required for pedestrians at intersections where complex signal phasing is used. The island width must be sufficient to meet the anticipated storage needs for pedestrians and bicyclists and/or disabled needs (i.e., wheelchairs) for refuge purposes.

28.7.2 Selection of Channelization

Channelization may consist of some combination of flush or raised, concrete or turf and triangular or elongated. Selection of an appropriate type of channelization will be determined on a case-by-case evaluation based on traffic characteristics, cost considerations and maintenance needs. The following sections offer guidance for channelization.

28.7.2.1 Flush Channelization

Flush channelization is appropriate in the following situations:

1. at locations requiring delineation of vehicular paths (e.g., major route turns, intersections with unusual geometry);

2. on high-speed rural highways to delineate separate turning lanes;

3. In restricted locations where vehicular path definition is desired, but space for larger, raised channelization is not available; and/or

4. to separate opposing traffic streams on low-speed streets.

28.7.2.2 Raised Channelization

Raised channelization emphasizes the location of the movement to be completed. Raised channelization is appropriate in the following situations:

1. at locations requiring positive delineation of vehicular paths (e.g., major route turns, intersections with unusual geometry);

2. where a primary function of an island is to provide a pedestrian refuge;

3. where a primary or secondary island function is the location of traffic signals, signs or other traffic control features;

4. where the channelization is intended to prohibit or prevent traffic movements; and/or

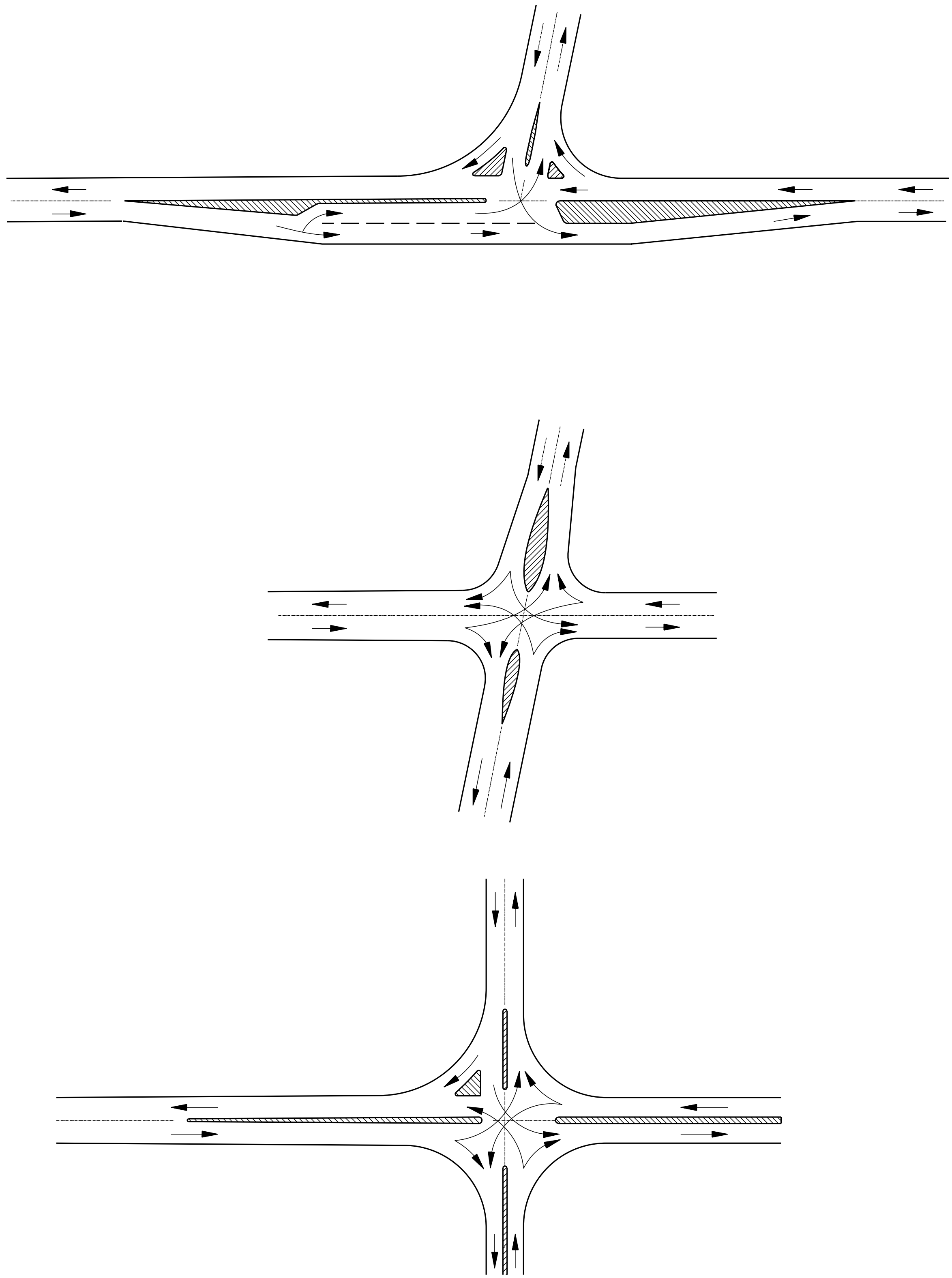
5. on low- to moderate-speed highways where the primary function is to separate high volumes of opposing traffic movements.

Where a raised channelization is selected, a sloping concrete curb will typically be used. In addition, the need for lighting the intersection will be determined on a case-by-case basis.

28.7.2.3 Medial Separators

Figure 28.7A illustrates several intersections with medial separators. Medial separators are advantageous in that they:

1. support the effective storage at stop bars,
2. provide better alignment and channelization for turning vehicles, and
3. improve intersection sight distances to the left for right-turning vehicles.



MEDIAL SEPARATORS

Figure 28.7A

28.7.3 Size

Traffic islands and medial separators should be large enough to command the driver's attention. Shapes and sizes will vary from one intersection to another. The following will apply:

1. Triangular Islands. The recommended minimum size is 75 ft^2 (7 m^2) (urban) and 100 ft^2 (10 m^2) (rural). Desirably, all triangular islands will be at least 100 ft^2 (10 m^2), if practical. Islands used for pedestrian refuge should be at least 150 ft^2 (15 m^2) to allow for the construction of curb ramps or channels for the disabled.

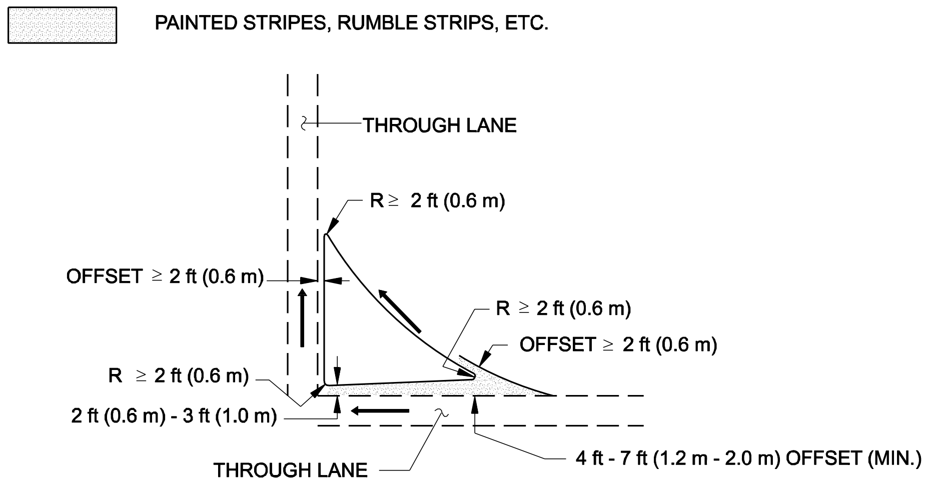
2. Medial Separators. The recommended minimum width is 6 ft (1.8 m). The desirable width is 10 ft (3 m).

28.7.4 Delineation

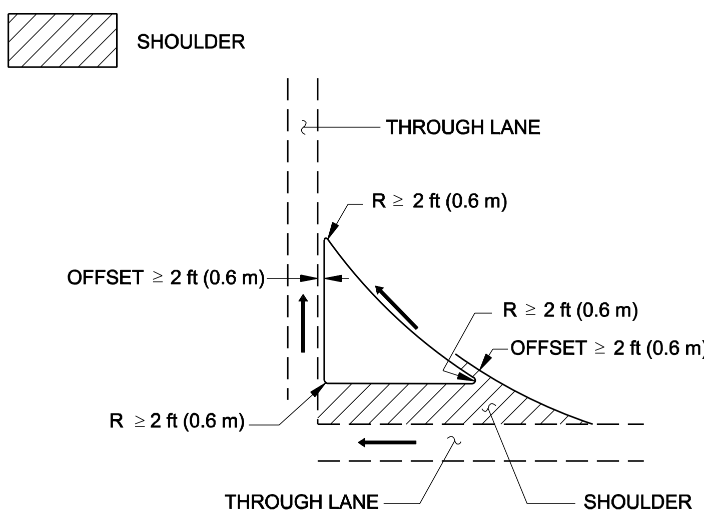
Pavement markings are used in advance of and around the raised channelization to warn the driver. These traffic control devices are especially important at the approach to divisional medial separators for the direction of approaching traffic.

28.7.5 Offset to Through Lanes

In urban areas on approaching roadways with curb offsets, offset the raised channelization at least 2 ft (0.6 m) from the travel lane (Figure 28.7B(a)). Where shoulders are present, offset raised channelization a distance equal to the shoulder width (Figure 28.7B(b)). In rural areas and where separate turn lanes are used, offset the island from the turning lane by at least 2 ft (0.6 m). If there are no turn lanes, the island should be offset a distance equal to the shoulder width.



(a) Curbed Island – No Shoulder



(b) Curbed Island with Shoulder

TRIANGULAR ISLANDS

Figure 28.7B

28.8 MEDIAN OPENINGS

28.8.1 Criteria/Spacing

28.8.1.1 Freeways

Turnarounds on rural freeways are normally provided where interchange spacing exceeds 5 mi (8 km) to avoid long travel distances for emergency, law enforcement and maintenance vehicles. Between interchanges, turnarounds are spaced at 3 mi to 4.0 mi (5 km to 6.5 km) intervals. They may be required at one or both ends of interchange facilities, depending on interchange type, to facilitate maintenance operations. Do not locate turnarounds closer than 1500 ft (450 m) to the end of an exit or entrance taper of a ramp or to any structure; see Figure 28.8A. Only locate turnarounds where above-minimum stopping sight distance and intersection sight distances are available and, preferably, not within curves requiring superelevation.

28.8.1.2 Non-Freeways

Median openings are provided on all divided highways with partial control of access or control by regulation provided that the openings are sufficiently spaced. Median openings are appropriate for the following situations:

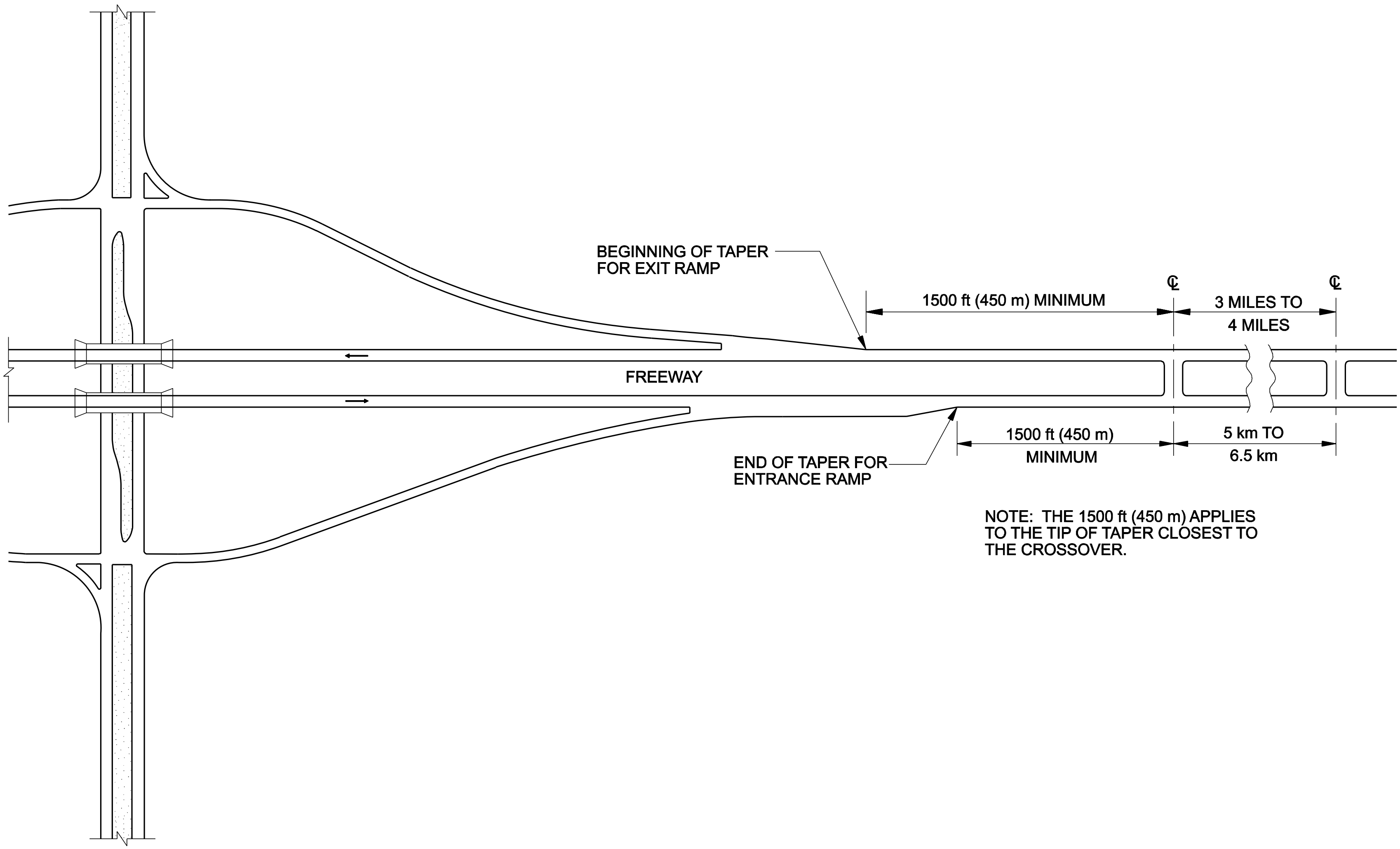
1. at most dedicated public streets (site specific),
2. for U-turn movements on long sections of a continuous raised median, or
3. at approaches serving major traffic generators.

Examples of major traffic generators include major shopping centers and special event facilities with several events per month. Small shopping plazas or single businesses are not considered major traffic generators.

In addition to the above criteria, median openings should be consistent with the following design considerations:

1. Signal Coordination. Median openings (both signalized and unsignalized) must not impair the traffic signal coordination of the overall facility.

2. Sight Distance. Do not locate median openings in areas of restricted sight distance (e.g., on a horizontal curve or near the apex of a crest vertical curve). Section 28.9 discusses the minimum intersection sight distances that should be available at a median opening.



LOCATIONS OF TURNAROUNDS ON FREEWAYS

Figure 28.8A

3. Turn-Lane Length. Median openings should only be provided if the full length of a left-turn lane can be provided and if the beginning of the turn-lane taper is at least 100 ft (30 m) from the median nose of the previous intersection. See the schematic in Figure 28.8B. The length of the left-turn lane will be determined by the criteria in Section 28.4.2.

4. Minimum Spacing. In no case may the number of median openings exceed three per 1000 ft (300 m).

28.8.2 Design

28.8.2.1 Turning Radii

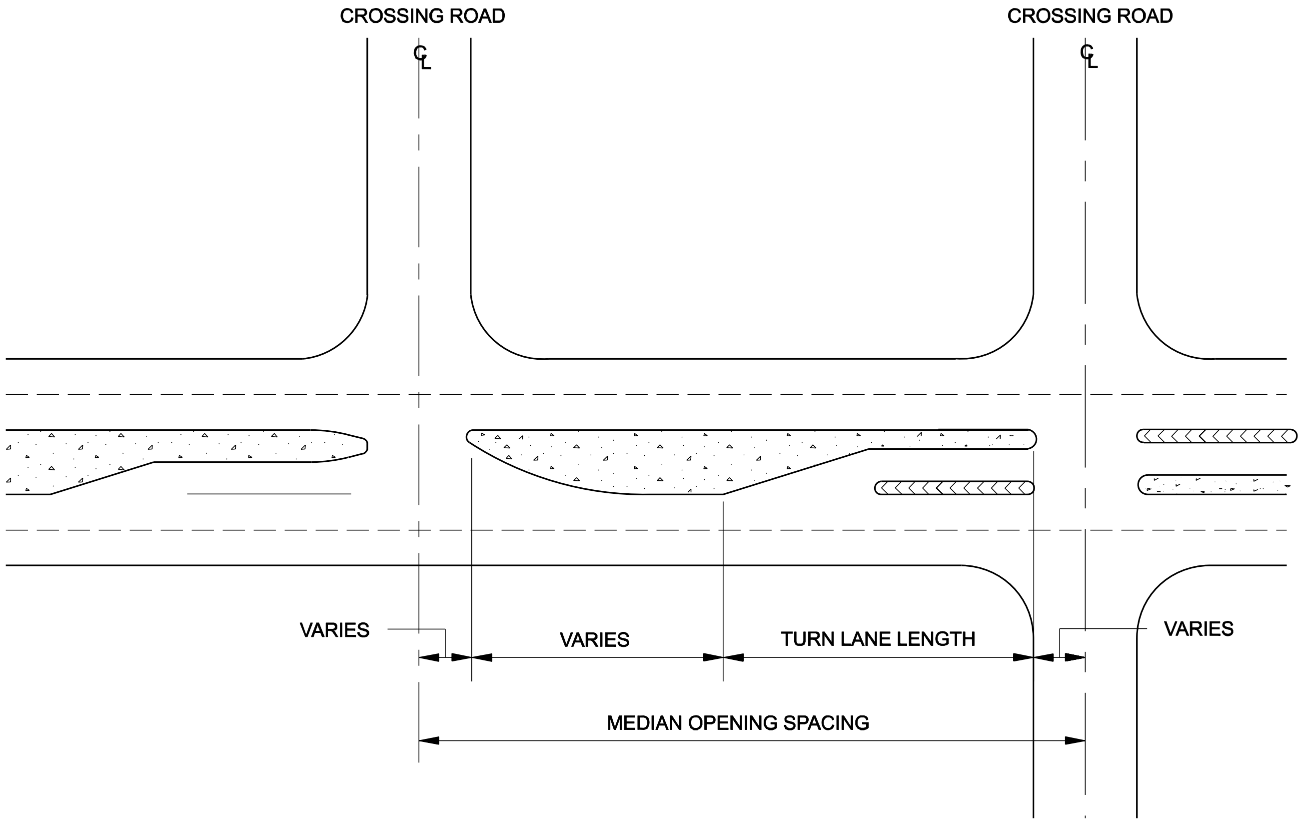
Median openings should be designed to properly accommodate left-turning vehicles. Turning radii for the intersection layout are based on the selection of the design vehicle, the turning characteristics of the design vehicle, the acceptable encroachment and the angle of turn. The discussion in Section 28.3 on these factors also applies to median openings.

28.8.2.2 Median End Design

The most common median end design is the semicircular end. This design is illustrated in Figure 28.8C and is used where the width of the divider is 4 ft to 10 ft (1.2 m to 3.0 m). The divider width (W_2) is the distance between the inside travel lanes or between the left-turn lane and the opposing inside travel lane. This is in contrast to the median width (W_1), which is measured between the edges of the two inside travel lanes and, therefore, includes the width of left-turn lanes, if present. When designing median ends, consider the following:

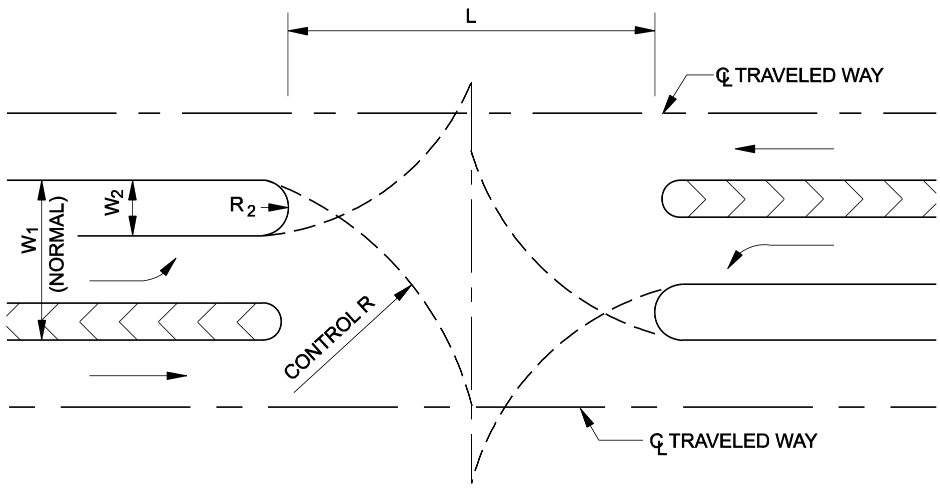
1. Offset Turn-Lanes. Where the median width is 18 ft to 75 ft (5.4 m to 22 m), the preferred practice is to provide an offset turn-lane design as discussed in Section 28.4.3. This alignment will force the vehicle waiting to make the turn as far to the left as practical, thereby maximizing the offset between the opposing left-turn lanes and providing improved visibility to the opposing through traffic.

2. Divider Width (W_2). The maximum practical width of m for the semicircular design is 10 ft (3 m). If m is larger than 10 ft (3 m), offset the left-turn lane until the divider width is 10 ft (3 m) or less. Desirably, "W_2" should be 4 ft (1.2 m) with any additional width placed to the right of the left-turn lane.



LOCATIONS OF MEDIAN OPENINGS ON NON-FREEWAYS

Figure 28.8B



Key: W_1 = median width measured between the two edges of the inside travel lanes, ft (m)

W_2 = width of divider (raised or flush) remaining after the widths of the left-turn lane (if any) and/or shoulders have been subtracted from the median width, ft (m)

Notes:

1. *The semicircular end is typically used where $W_2 \leq 10$ ft (3 m).*
2. *Control R is dependent on the turning radii of the design vehicle selected.*
3. *$R_2 = W_2/2$*

MEDIAN OPENINGS
(Semicircular End)

Figure 28.8C

3. No Turn Lanes. For those medians without a left-turn lane (e.g., T-intersections, ramp terminals), the median end shape may be a bullet or half-bullet nose. Design criteria for bullet noses can be found in the AASHTO A Policy on Geometric Design of Highways and Streets.

4. Wide Medians. Where the medians are wider than 75 ft (22 m), treat the design as two separate intersections.

28.8.2.3 Length of Opening

The length of a median opening should properly accommodate the turning path of the design vehicle. The length of opening for any divided highway should be equal to or greater than the width of the intersecting road, but not less than 40 ft (12 m). Each median opening will be evaluated individually to determine its proper design. The designer should consider the following factors in the evaluation:

1. Turning Templates. The designer should check the proposed design with the turning templates or with a computer-turning template program for the design vehicles likely to use the intersection. Give consideration to the frequency of the turn and to the encroachment onto adjacent travel lanes or shoulders by the turning vehicle.

2. Nose Offset. At 4-leg intersections, traffic passing through the median opening (going straight) will pass the nose of the median end. To provide a sense of comfort for these drivers, the offset between the nose end and through travel lane (extended) should be at least 2 ft (0.6 m).

3. Intersection Symmetry. The 3 or 4 legs of the intersection may have various combinations of through travel lanes and turning lanes. The designer should give consideration to the overall symmetry of the intersection. This, in turn, may affect the design of the median opening.

4. Location of Crosswalks. Desirably, pedestrian crosswalks will intersect the median to provide some refuge for pedestrians. Therefore, the median opening design should be coordinated with the location of crosswalks.

28.8.2.4 U-Turns

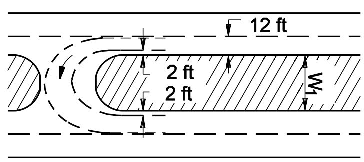
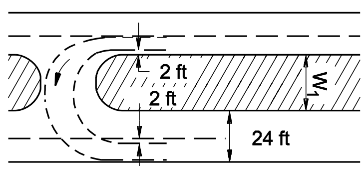
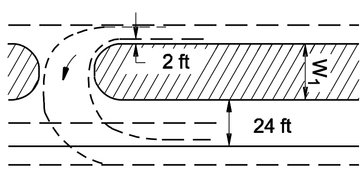
As discussed in Section 28.8.1, median openings sometimes may need to be designed to accommodate U-turns on divided highways. Preferably, a vehicle should be able to begin and end the U-turn on the inner lanes next to the median. This design also allows a stopped vehicle to be fully protected within the median area. The sight distance should be adequate to provide for the unexpected U-turn maneuver on an access-controlled highway. Figure 28.8D provides the minimum recommended median widths for U-turn maneuvers and various design vehicles.

28.8.2.5 Special Designs

At certain intersections, special designs may be required for the median opening. This may include the following:

1. Skewed Intersections. Preferably, median crossovers should not be skewed. Skewed median openings require larger openings which may require special channelization and other adjustments to the left-turn lanes. Each skewed design should be designed individually to determine the appropriate layout.

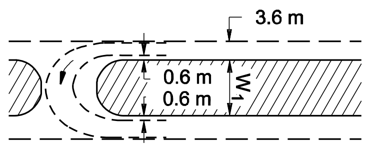
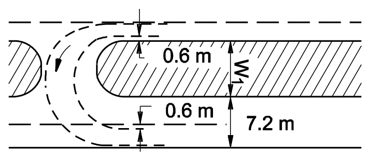
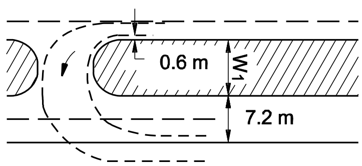
2. Movement Prohibition. Where traffic volumes are significant, it may be necessary to prohibit left turns from the side street. However, it still may be desirable to permit a left turn from the through streets. This design will be determined and designed on a case-by-case basis.

Type of Maneuver		M - Min. width of median for design vehicle (ft)				
		P	SU	BUS	WB-40	WB-50
		Length of design vehicle (ft)				
		19	30	40	50	55
Inner Lane to Inner Lane		30	63	63	61	71
Inner Lane to Outer Lane		18	51	51	49	59
Inner Lane to Shoulder		8	41	41	39	49

Note: The selected design vehicle will affect the length of the median opening.

MINIMUM WIDTHS NEEDED FOR U-TURNS
(US Customary)

Figure 28.8D

Type of Maneuver		W_1 - Min. Width of Median (m) for Design Vehicle				
		P	SU	BUS	WB-12	WB-15
		Length of Design Vehicle (m)				
		5.7	9.0	12.0	15.0	16.5
Inner Lane to Inner Lane		9	19	19	18	21
Inner Lane to Outer Lane		5	15	15	15	18
Inner Lane to Shoulder		2	12	12	12	15

MINIMUM DESIGNS FOR U-TURNS
(Metric)

Figure 28-8D

28.9 INTERSECTION SIGHT DISTANCE (ISD)

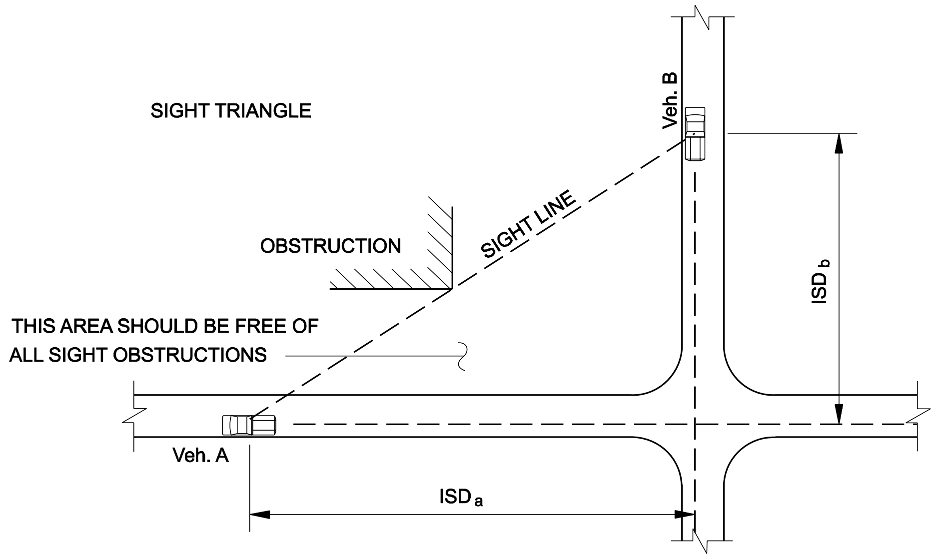
For an at-grade intersection to operate properly, adequate sight distance should be available. The designer should provide sufficient sight distance for a driver to perceive potential conflicts and to perform the actions needed to negotiate the intersection safely. The additional costs and impacts of removing sight obstructions are often justified. If it is impractical to remove an obstruction blocking the sight distance, the designer should consider providing traffic control devices or design applications (e.g., warning signs, turn lanes) that may not otherwise be considered.

In general, ISD refers to the corner sight distance available in intersection quadrants that allows a driver approaching an intersection to observe the actions of vehicles on the crossing leg(s). ISD evaluations involve establishing the needed sight triangle in each quadrant by determining the legs of the triangle on the two crossing roadways. The necessary clear sight triangle is based on the type of traffic control at the intersection and on the design speeds of the two roadways.

The Department uses gap acceptance as its basic concept in the design of intersection sight distance. This gap acceptance design is based on the criteria and theory presented in AASHTO's A Policy on Geometric Design of Highways and Streets.

28.9.1 Intersections With No Control

Intersections between low-volume and low-speed roads/streets may have no traffic control. At these intersections, sufficient corner sight distance should be available to allow approaching vehicles to adjust their speed to avoid a collision, typically 50% of their mid-block running speed. Figure 28.9A provides the ISD criteria for intersections with no traffic control. For approach grades greater than 3%, adjust the ISD values obtained in Figure 28.9A with the applicable ratios in Figure 28.9B.



US Customary					
Design Speed (mph)	15	20	25	30	35
*Intersection Sight Distance (ft)	70	90	115	140	165
Metric					
Design Speed (km/h)	20	30	40	50	60
*Intersection Sight Distance (m)	20	25	35	45	55

Note: For approach grades greater than 3%, multiply the sight distance values in this table by the appropriate adjustment factor from Figure 28.9B. The grade adjustment is based on the approach roadway grade only.

Example

Given: No traffic control at intersection

Design speed — 35 mph (Highway A)
25 mph (Highway B)

Problem: Determine legs of sight triangle.

Solution: From above table — ISD_a = 165 ft
ISD_b = 115 ft

INTERSECTION SIGHT DISTANCE
(No Traffic Control)

Figure 28.9A

US Customary											
Approach Grade (%)	Design Speed (mph)										
	20	25	30	35	40	45	50	55	60	65	70
-6	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2
-5	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2
-4	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Metric										
Approach Grade (%)	Design Speed (km/h)									
	30	40	50	60	70	80	90	100	110	120
-6	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2
-5	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2
-4	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Note: Based on ratio of stopping sight distance on specified approach grade to stopping sight distance on level terrain. The grade adjustment is based on the approach roadway grade only.

ADJUSTMENT FACTORS FOR APPROACH SIGHT DISTANCE BASED ON APPROACH GRADE

Figure 28.9B

28.9.2 Stop Controlled/Traffic-Signal Controlled

Where traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position assuming that the approaching vehicle comes into view as the stopped vehicle begins its departure.

The stopped-controlled criteria required will also apply to a signalized intersection. This is reasonable because of the increased driver work load at intersections and the potential conflicts involved when vehicles turn onto or cross the highway. These include:

1. violation of the signal,
2. right-turns-on-red,
3. signal malfunction, and/or
4. use of flashing yellow/red mode during part of the day.

If these criteria cannot be met, give consideration to prohibiting right-turn-on-red at the intersection or prohibiting the flashing mode. This determination will be based on field investigations and will be determined on a case-by-case basis.

28.9.2.1 Basic Criteria

The Department uses gap acceptance as the conceptual basis for its intersection sight distance (ISD) criteria at stop-controlled and traffic-signal controlled intersections. The intersection sight distance is obtained by providing clear sight triangles both to the right and left as shown in Figure 28.9C. The length of legs of these sight triangles are determined as follows:

1. Minor Road. The length of leg along the minor road is based on two parts. The first is the location of the driver's eye on the minor road. This is typically assumed to be 15 ft (4.5 m) from the edge of traveled way for the major road and in the center of the lane on the minor road; see Figure 28.9C. The second part is based on the distance to the center of the vehicle on the major road. For right-turning vehicles, this is assumed to be the center of the closest travel lane from the left. For left-turning vehicles, this is assumed to be the center of the closest travel lane for vehicles approaching from the right; see Figure 28.9C.

2. Major Road. The length of the sight triangle leg or ISD along the major road is determined using the following equation:

$ISD = 1.47\ V_{major}\ t_g$ (US Customary) (Equation 28.9-1)

$ISD = 0.278\ V_{major}\ t_g$ (Metric) (Equation 28.9-1)

Where:

ISD = length of sight triangle leg along major road, ft (m)

V_{major} = design speed of major road, mph (km/h)

t_g = the gap for entering the major road, sec

The time gap (t_g) varies according to the design vehicle, the grade on the minor road approach, the number of lanes on the major roadway, the type of operation and the intersection skew. Section 28.9.2.5 presents several examples on the application of ISD.

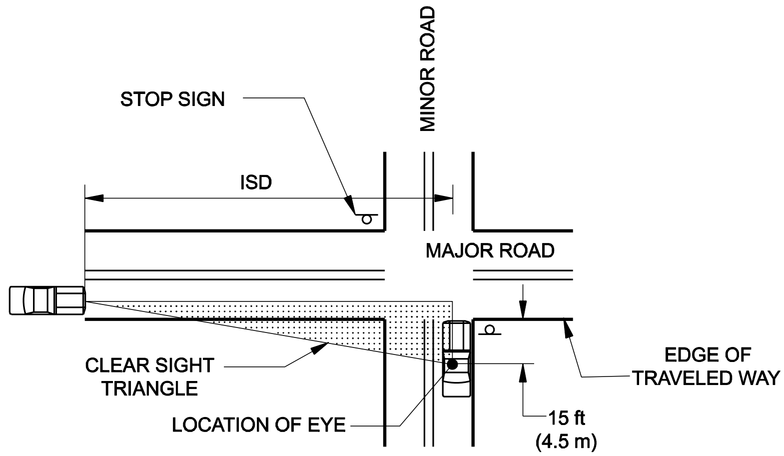
3. Trucks. At some intersections (e.g., near truck stops, interchange ramps, grain elevators), the designer may want to use the truck as the design vehicle for determining the ISD. The gap acceptance times (t_g) for single-unit and tractor/ semitrailer trucks are provided in Figures 28.9D and 28.9G. ISD values for level, 2-lane roadways are presented in Figures 28.9E and 28.9H.

4. Height of Eye/Object. The height of eye for passenger cars is assumed to be 3.5 ft (1080 mm) above the surface of the minor road. The height of object (approaching vehicle on the major road) is also assumed to be 3.5 ft (1080 mm). An object height of 3.5 ft (1080 mm) assumes that a sufficient portion of the oncoming vehicle must be visible to identify it as an object of concern by the minor road driver. If there is a sufficient number of trucks to warrant their consideration, assume an eye height of 7.6 ft (2330 mm). If a truck is the assumed entering vehicle, the object height will still be 3.5 ft (1080 mm) for the passenger car on the major road.

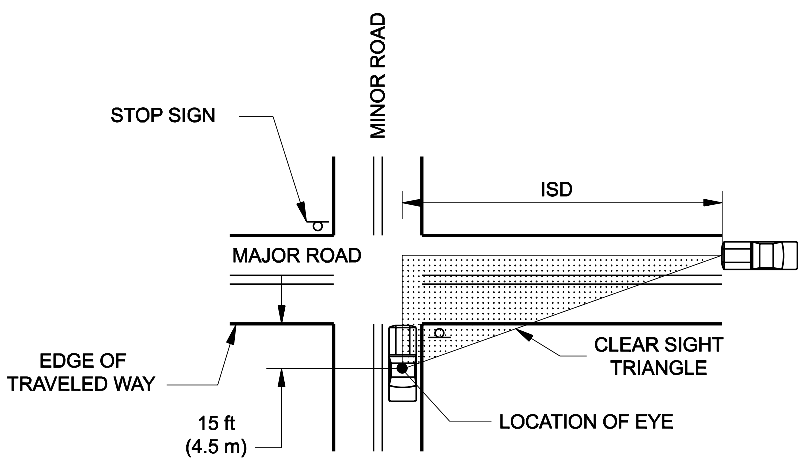
5. Skew. At skewed intersections where the intersection angle is less than 60°, adjustments may need to be made to account for the extra distance the vehicle needs to travel across opposing lanes. Using the procedures discussed in Comment #2 in Section 28.9.2.2 and/or Section 28.9.2.4, determine the appropriate ISD value based on this extra travel distance.

6. Examples. For examples on the application of ISD, see Section 28.9.2.5.

Within this clear sight triangle, if practical, the objective is to remove, lower objects or trim lower branches that obstruct the driver’s view. These objects may include buildings, parked or turning vehicles, trees, hedges, tall crops, unmowed grass, fences, retaining walls and the existing ground line. In addition, where an interchange ramp



CLEAR SIGHT TRIANGLE FOR VIEWING TRAFFIC APPROACHING FROM THE LEFT



CLEAR SIGHT TRIANGLE FOR VIEWING TRAFFIC APPROACHING FROM THE RIGHT

CLEAR SIGHT TRIANGLE (STOP-CONTROLLED) INTERSECTIONS

Figure 28.9C

intersects the major road or crossroad near a bridge on a crest vertical curve, objects (e.g., bridge parapets, piers, abutments, the crest vertical curve itself) may restrict the clear sight triangle.

28.9.2.2 Left-Turn From the Minor Road

To determine the intersection sight distance for vehicles turning left onto the major road, the designer should use Equation 28.9-1 and the time gap (t_g) presented in Figure 28.9D. Figure 28.9E, which solves Equation 28.9-1, provides the ISD values for all design vehicles on 2-lane, level facilities. The designer should also consider the following:

1. Multilane Facilities. For multilane facilities, the gap acceptance times presented in Figure 28.9D should be adjusted to account for the additional distance required by the turning vehicle to cross the additional lanes or median. For left turns onto multilane highways, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, in excess of one, to be crossed by the turning vehicle. Assume that the left-turning driver will enter the left travel lane on the far side of the major road. For example, the gap acceptance time for a passenger car turning left onto an undivided six-lane facility would be 7.5 seconds plus 0.5 seconds for each of the two additional lanes needed to be crossed. The total gap time required is, therefore, 8.5 seconds.

2. Medians. For a multilane facility that does not have a median wide enough to store a stopped vehicle, divide the median width by 12 ft (3.6 m) to determine the corresponding number of lanes, and then use the criteria in Comment #1 above to determine the appropriate time factor.

   On multilane facilities with a median wide enough to store the stopped vehicle, the designer should evaluate the move in two steps; see Figure 28.9F:

   Step 1. With the vehicle stopped on the minor road (the bottom portion in Figure 28.9F), use the gap acceptance times and distances for a vehicle turning right (Figures 28.9G and 28.9H) to determine the applicable ISD. Note that this is also the same distance as the crossing maneuver; see Sections 28.9.2.3 and 28.9.2.4.

   Step 2. With the vehicle stopped in the median (top portion in Figure 28.9F), assume a two-lane roadway design and use the gap acceptance times and distances for vehicles turning left (Figures 28.9D and 28.9E) to determine the applicable ISD.

3. Approach Grades. If the approach grade on the minor road exceeds +3%, multiply the percent grade on the approach by 0.2 and add this to the base gap acceptance times in Figure 28.9D. Use the adjusted t_g in Equation 28.9-1 to determine the applicable ISD. Do not apply the grade adjustment if the approach grade is negative.

28.9.2.3 Right Turn From the Minor Road

To determine the intersection sight distance for vehicles turning right onto the major road, the designer should use Equation 28.9-1 and the time gap (t_g) presented in Figure 28.9G. Figure 28.9H, which solves Equation 28.9-1, provides the ISD values for all design vehicles on 2-lane, level facilities. The designer should also consider the following:

1. Approach Grades. If the approach grade on the minor road exceeds +3%, multiply the percent grade on the approach by 0.1 and add this to the base gap acceptance time. Use the adjusted t_g in Equation 28.9-1 to determine the applicable ISD. Do not apply the grade adjustment if the approach grade is negative.

2. Multilane Facilities. Because the turning vehicle is assumed to be turning into the nearest right through lane, no adjustments to the gap times are required.

28.9.2.4 Straight Through Crossing Vehicle

In the majority of cases, the intersection sight distance for turning vehicles typically will provide adequate sight distance to allow a vehicle to cross the major road. However, in the following situations, the crossing sight distance may be the more critical movement:

1. where left and/or right turns are not permitted from a specific approach and the crossing maneuver is the only legal or expected movement (e.g., indirect left turns);

2. where the design vehicle must cross more than six travel lanes or, with medians, the equivalent distance; or

3. where a substantial volume of heavy vehicles cross the highway and there are steep grades on the minor road approach.

Design Vehicle	Gap Acceptance Time (t_g) (sec)
Passenger Car	7.5
Single-Unit Truck	9.5
Combination Truck	11.5

Note: See Section 28.9.2.2 for any necessary adjustments.

GAP ACCEPTANCE TIMES FOR STOPPED-CONTROLLED INTERSECTIONS
(Left Turn from Minor Road)

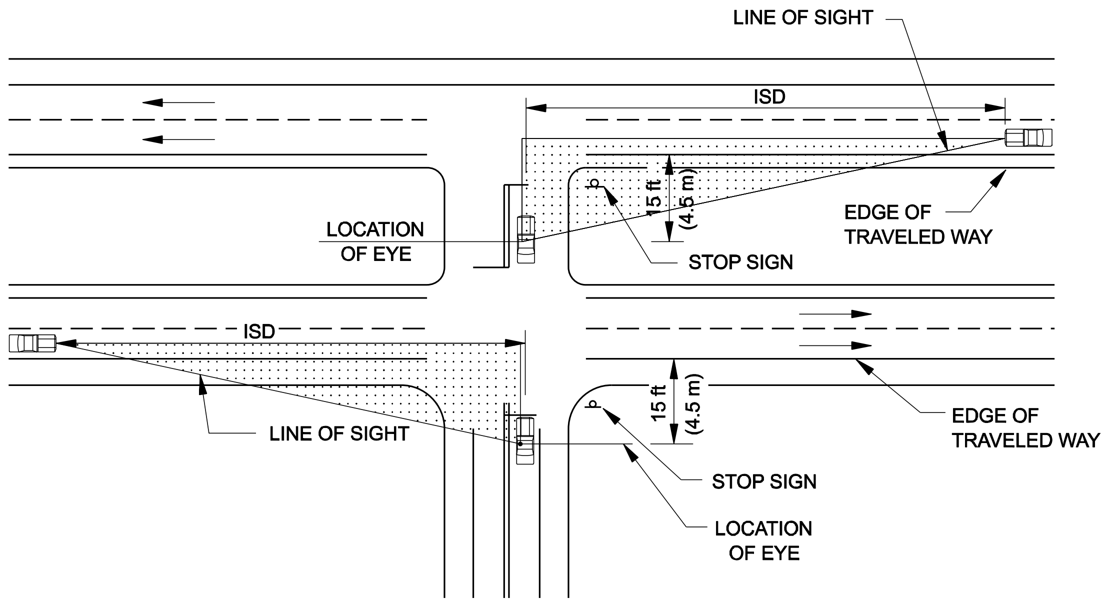
Figure 28.9D

US Customary			
Design Speed (V_{major}) (mph)	ISD (ft)		
	Passenger Cars	Single-Unit Trucks	Tractor/Semitrailers
20	225	280	340
25	280	350	425
30	335	420	510
35	390	490	595
40	445	560	680
45	500	630	765
50	555	700	850
55	610	770	930
60	665	840	1015
65	720	910	1100
70	775	980	1185
Metric			
Design Speed (V_{major}) (km/h)	ISD (m)		
	Passenger Cars	Single-Unit Trucks	Tractor/Semitrailers
30	65	80	95
40	85	105	130
50	105	130	160
60	130	160	190
70	150	185	225
80	170	210	255
90	190	240	290
100	210	265	320
110	230	290	350

Note: These ISD values assume a minor road approach grade $\leq$ +3%.

TWO-LANE INTERSECTION SIGHT DISTANCES
(Left Turn From Minor Road)

Figure 28.9E



INTERSECTION SIGHT DISTANCE
(Divided Facilities)

Figure 28.9F

Design Vehicle	Gap Acceptance Time (t_g) (sec)
Passenger Car	6.5
Single-Unit Truck	8.5
Combination Truck	10.5

Adjustments:

1. *<u>Multilane Highway</u>. Where the design vehicle is crossing a major road with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane in excess of two. See the discussion in* Section 28.9.2.2 *for additional guidance.*

2. *<u>Approach Grade</u>. If the approach grade on the minor road exceeds +3%, multiply the percent grade of the minor road approach by 0.1 seconds and add this to the gap acceptance times in the table above.*

GAP ACCEPTANCE TIMES
(Right Turn from Minor Road and Crossing Major Road)

Figure 28.9G

US Customary			
Design Speed (V_{major}) (mph)	ISD (ft)		
	Passenger Cars	Single-Unit Trucks	Tractor/Semitrailers
20	195	250	310
25	240	315	390
30	290	375	465
35	335	440	545
40	385	500	620
45	430	565	695
50	480	625	775
55	530	690	850
60	575	750	930
65	625	815	1005
70	670	875	1080
Metric			
Design Speed (V_{major}) (km/h)	ISD (m)		
	Passenger Cars	Single-Unit Trucks	Tractor/Semitrailers
30	55	75	90
40	75	95	120
50	95	120	150
60	110	145	180
70	130	170	205
80	145	190	235
90	165	215	265
100	185	240	295
110	200	560	325

Note: These ISD values assume a minor road approach grade ≤ +3%.

TWO-LANE INTERSECTION SIGHT DISTANCES
(Right Turn From Minor Road or Crossing Major Road)

Figure 28.9H

Use Equation 28.9-1 and the gap acceptance times (t_g) and the adjustment factors in Figure 28.9G to determine the ISD for crossing maneuvers. Where medians are present, include the median width in the overall length to determine the applicable time gap. Divide this width by 12 ft (3.6 m) to determine the corresponding number of lanes for the crossing maneuver.

28.9.2.5 Examples of ISD Applications

The following three examples illustrate the application of the ISD criteria:

Example 28-1

Given: Minor road intersects a 4-lane highway with a TWLTL.
Minor road is stop controlled.
Design speed of the major highway is 50 mph.
All travel lane widths are 12 ft.
The TWLTL width is 14 ft.
Trucks are not a concern.

Problem: Determine the intersection sight distance to the left and right from the minor road.

Solution: The following steps will apply:

1. For the vehicle turning right, the ISD to the left can be determined directly from Figure 28.9H. For the 50 mph design speed, the ISD to the left is 480 ft.

2. For the vehicle turning left, the ISD must reflect the additional time required to cross the additional lanes; see Comment #1 in Section 28.9.2.2. The following will apply:

    a. First, determine the extra width required by the one additional travel lane and the TWLTL and divide this number by 12 ft:

    $$\frac{(12+14)}{12} = 2.2 \text{ lanes}$$

    b. Next, multiply the number of lanes by 0.5 seconds to determine the additional time required:

    (2.2 lanes)(0.5 sec/lane) = 1.1 seconds

c. Add the additional time to the basic gap time of 7.5 seconds and insert this value into Equation 28.9-1:

ISD = (1.47)(50)(7.5 + 1.1) = 632 ft

Provide an ISD of 635 ft to the right for the left-turning vehicle.

3. Check the crossing vehicle, as discussed in Section 28.9.2.4. The following will apply:

a. First determine the extra width required by the two additional travel lanes and the TWLTL and divide this number by 12 ft:

$$\frac{(12 + 12 + 14)}{12} = 3.2 \text{ lanes}$$

b. Next, multiply the number of lanes by 0.5 seconds to determine the additional time required.

(3.2 lanes)(0.5 sec/lane) = 1.6 seconds

c. Add the additional time to the basic gap time of 6.5 seconds and insert this value into Equation 28.9-1:

ISD = (1.47)(50)(6.5 + 1.6) = 595 ft

The 595 ft for the crossing maneuver is less than the 635 ft required for the left-turning vehicle and, therefore, is not the critical maneuver.

Example 28-2

Given: Minor road intersects a 4-lane divided highway.
Minor road is stop-controlled.
Design speed of the major highway is 55 mph.
All travel lane widths are 12 ft.
The median width is 100 ft.
Trucks are not a concern.

Problem: Determine the intersection sight distance to the left and right from the minor road.

Solution: The following steps apply:

1. For the vehicle turning right, the ISD to the left can be determined directly from Figure 28.9H. For the 55 mph design speed, the ISD to the left is 530 ft.

2. Determine if the crossing maneuver is critical; see Section 28.9.2.4. No adjustments are required to the base time of 6.5 seconds. Therefore, the cross maneuver is the same as the right-turning vehicle.

3. For the vehicle turning left, assume the passenger car is stopped in the median; see Figure 28.9F. The ISD to the right can be determined directly from Figure 28.9E. For the 55 mph design speed, the ISD to the right is 610 ft. The crossing maneuver will not be critical.

Example 28-3

Given: Minor road intersects a 2-lane highway.
Minor road is stop controlled.
Design speed of the major highway is 55 mph.
All travel lane widths are 12 ft.
The approach grade on the minor road is 4.5%.
Tractor/semitrailer trucks are a concern.

Problem: Determine the intersection sight distance to the left and right from the minor road.

Solution: The following steps will apply:

1. For the left-turning vehicle, the base gap acceptance time from Figure 28.9D is 11.5 seconds. Add the additional time due to the approach grade (0.2 seconds per percent grade) to the base gap time; see Comment #3 in Section 28.9.2.2:

   $(0.2)(4.5) + 11.5 = 12.4$ seconds

   Then, using Equation 28.9-1:

   $S = (1.47)(55)(12.4) = 1002$ ft

2. The ISD for the right-turning vehicle is determined similarly:

   $(0.1)(4.5) + 10.5 = 11$ seconds

   Then, using Equation 28.9-1:

ISD = (1.47)(55)(11.0) = 890 ft

3. The crossing maneuver will not be critical.

28.9.3 Yield Control

At intersections controlled by a yield sign, drivers on the minor road will typically:

1. slow down as they approach the major road, typically to 60% of the approach speed;
2. based on their view of the major road, make a stop/continue decision; and
3. either brake to a stop or continue their crossing or turning maneuver onto the major road.

Yield control criteria is based on a combination of the no control ISD discussed in Section 28.9.1 and the stop-controlled ISD as discussed in Section 28.9.2. To determine the applicable clear sight triangles of the approaches, the following will apply; see Figure 28.9I:

1. Crossing Maneuver. Use the following to determine the legs of the clear sight triangle; Illustration a in Figure 28.9I:

    a. Minor Road. The leg on the minor road approach can be determined directly from Figure 28.9J.

    b. Major Road. The leg on the major road is determined using the following equations and the times listed in Figure 28.9J:

$$t_g = t_a + \frac{w + L_a}{0.88\ V_{minor}}$$ (US Customary) (Equation 28.9-2)

$$t_g = t_a + \frac{w + L_a}{0.0167\left(V_{minor}\right)}$$ (Metric) (Equation 28.9-2)

$$b = 1.47\ V_{major}\ t_g$$ (US Customary) (Equation 28.9-1)

$$b = 0.278\left(V_{major}\right)\left(t_g\right)$$ (Metric) (Equation 28.9-1)

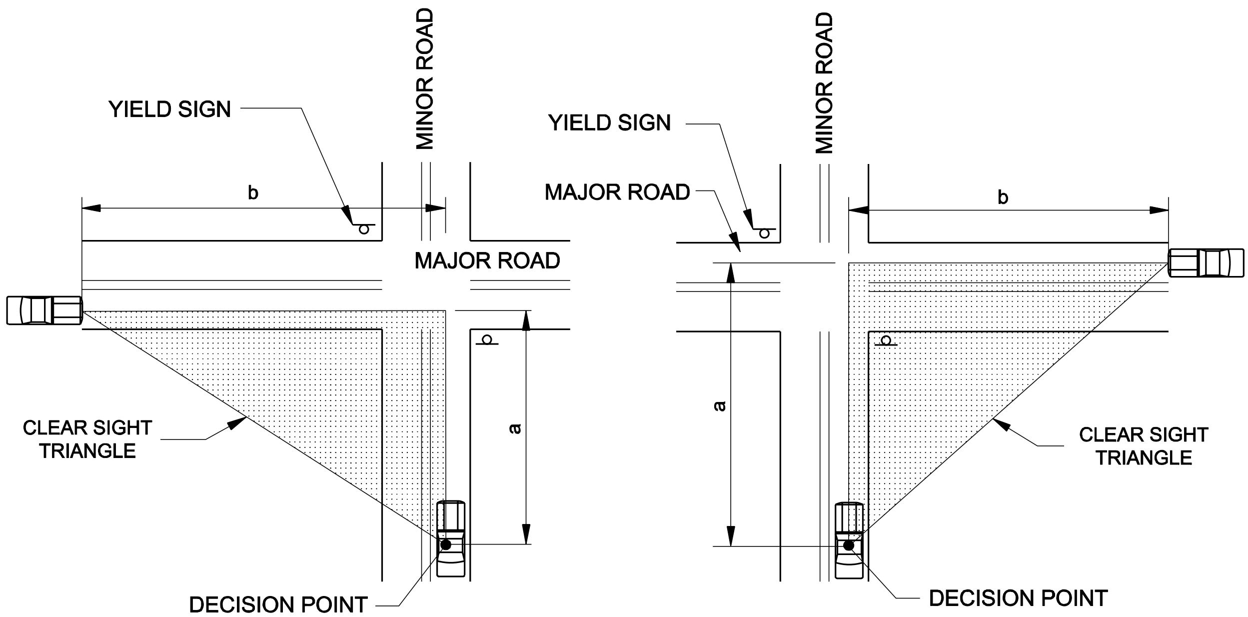
Where:

- b = length of leg of sight triangle along the major road, ft (m)
- t_g = travel time to reach and clear the major road in a crossing maneuver, sec
- t_a = travel time to reach the major road from the decision point for a vehicle that does not stop (sec) (use appropriate value for the minor-road design speed from Figure 28.9J, adjusted for approach grade, where appropriate)
- w = width of intersection to be crossed, ft (m)
- L_a = length of design vehicle, ft (m)
- V_{minor} = design speed of minor road, mph (km/h)
- V_{major} = design speed of major road, mph (km/h)

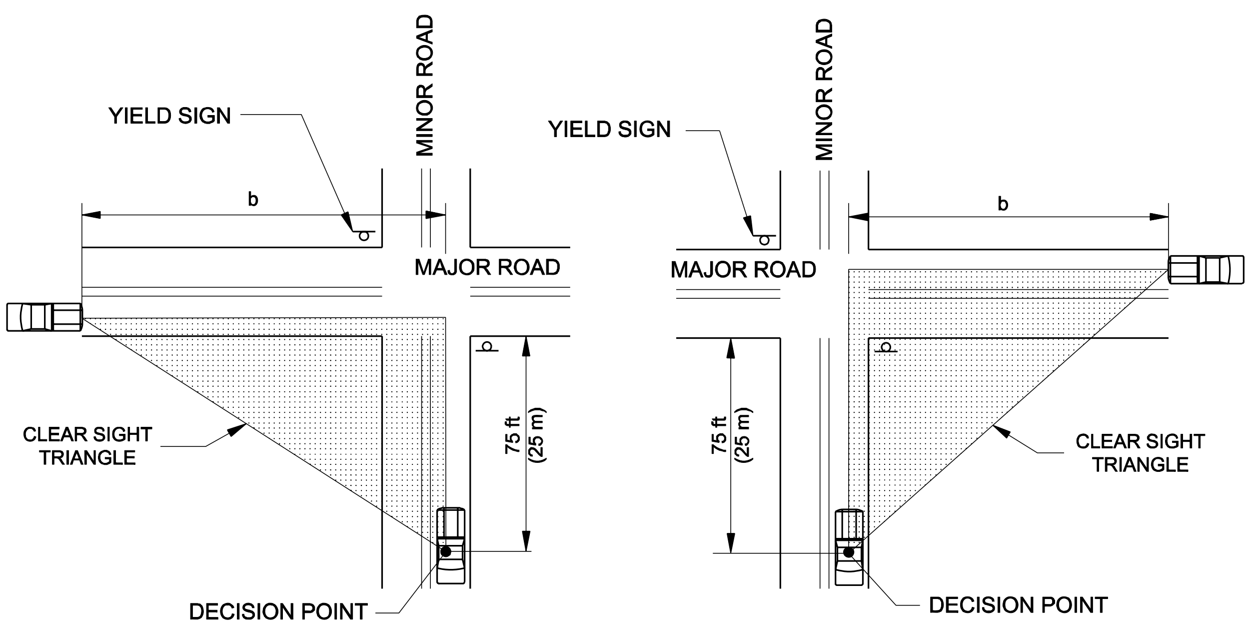
2. Turning Maneuvers. For the turning left or right vehicle, the approach legs are determined as follows; see Illustration b in Figure 28.9I:

   a. Minor Road. The assumed turning speed from the minor road to the major road is 10 mph (15 km/h). This corresponds to an approach distance of 75 ft (25 m) along the minor road leg.

   b. Major Road. To determine the legs along the major road, use the same procedures as discussed in Section 28.9.2.2 for the stop-controlled intersection, Equation 28.9-1 and the critical gap times listed in Figure 28.9K. Because the critical gap times are longer than the stop-controlled gap times, it will be unnecessary to determine the sight distance criteria for the vehicle which stops at the yield sign.



a) CROSSING MANEUVERS



b) TURNING MANEUVERS

INTERSECTION SIGHT DISTANCE APPLICATION
(Yield Control)

Figure 28.9I

US Customary		
Design Speed (mph)	Approach Distance Along Minor Road[1](a) (ft)	Travel Time From Decision Point to Major Road[1][2] (t_a) (sec)
20	100	3.7
25	130	4.0
30	160	4.3
35	195	4.6
40	235	4.9
45	275	5.2
50	320	5.5
55	370	5.8
60	420	6.1
65	470	6.4
70	530	6.7
Metric		
Design Speed (km/h)	Approach Distance Along Minor Road[1](a) (m)	Travel Time From Decision Point to Major Road[1][2] (t_a) (sec)
30	30	3.6
40	40	4.0
50	55	4.4
60	65	4.8
70	80	5.1
80	100	5.5
90	115	5.9
100	135	6.3
110	155	6.7

(1) For minor-road approach grades that exceed 3%, multiply by the appropriate adjustment factor from Figure 28.9B. Do not apply the adjustment factor to approaches with negative grades.

(2) Travel time applies to a vehicle that slows before crossing the intersection but does not stop.

ISD ASSUMPTIONS FOR YIELD CONTROLLED INTERSECTION
(Crossing Maneuver)

Figure 28.9J

Design Vehicle	Gap Acceptance Time (t_g)(sec)
Passenger Car	8.0
Single-Unit Truck	10.0
Combination Truck	12.0

Adjustments:

If the approach grade on the minor road exceeds 3%, the following applies:

1. *For right turns, multiply the percent grade of the minor road approach by 0.1 and add it to the base gap acceptance time.*

2. *For left turns, multiply the percent grade of the minor road approach by 0.2 and add it to the base gap acceptance time.*

GAP ACCEPTANCE TIMES FOR YIELD CONTROL INTERSECTIONS
(Turning Maneuvers)

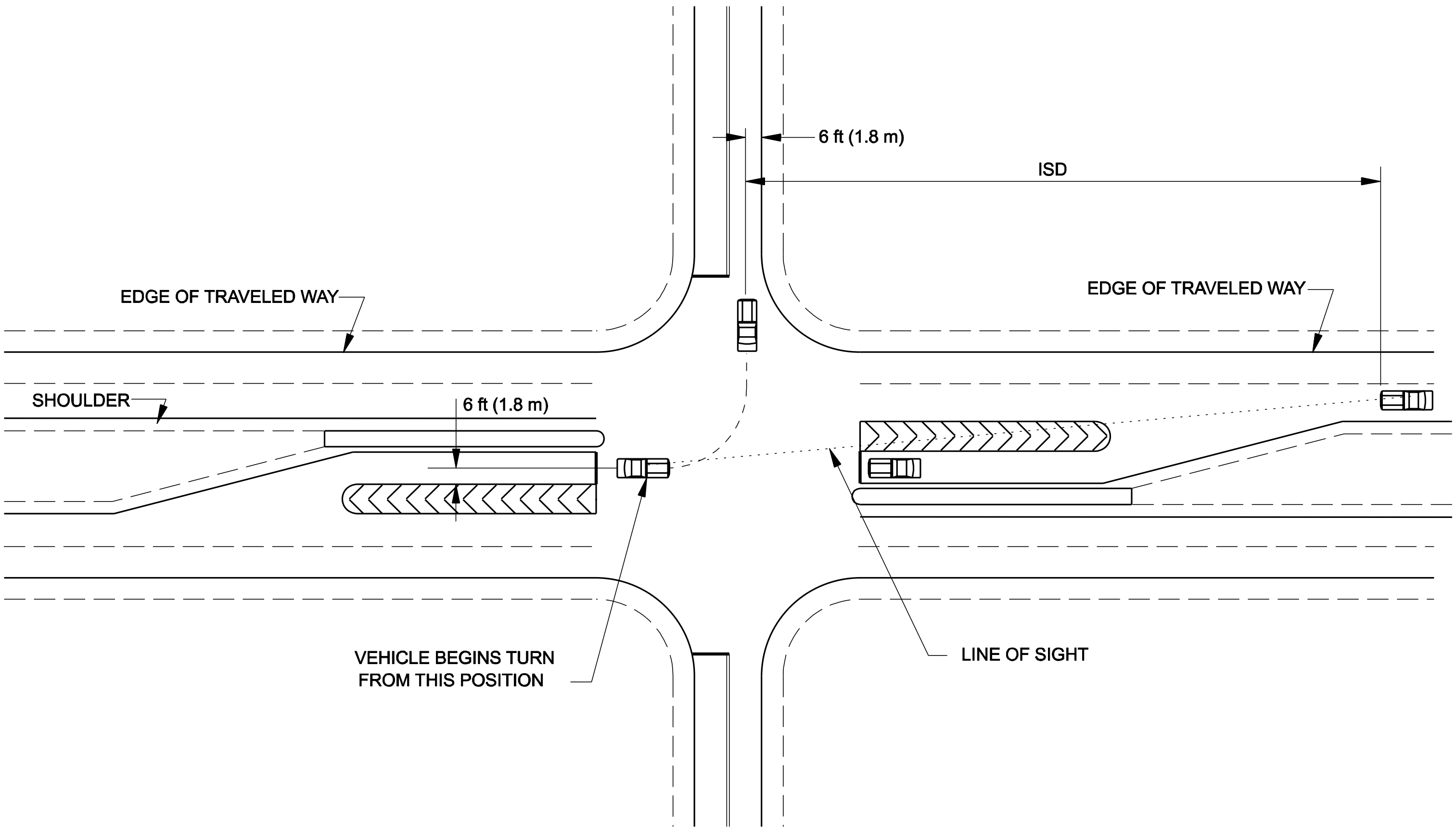
Figure 28.9K

28.9.4 All-Way Stop

At intersections with all-way stop control, provide sufficient sight distance so that the first stopped vehicle on each approach is visible to all other approaches. The ISD criteria for left- or right-turning vehicles as discussed in Section 28.9.2 are not applicable in this situation. Often, intersections are converted to all-way stop control to address limited sight distance at the intersection. Therefore, providing additional sight distance at the intersection is unnecessary.

28.9.5 Stopped Vehicle Turning Left

At all intersections, regardless of the type of traffic control, the designer should consider the sight distance needs for a stopped vehicle turning left from the major road. This is illustrated in Figure 28.9L. The driver must see straight ahead for a sufficient distance to turn left and clear the opposing travel lanes before an approaching vehicle reaches the intersection. In general, if the major highway has been designed to meet the stopping sight distance criteria, intersection sight distance only will be a concern where the major road is on a horizontal curve, where there is a median or where there are opposing vehicles making left turns at an intersection.



Notes:

1. *See [Figure 28.9N] for ISD values.*
2. *See [Section 28.9.5] for discussion and application.*

INTERSECTION SIGHT DISTANCE FOR A STOPPED VEHICLE TURNING LEFT
(On Major Road)

Figure 28.9L

Use Equation 28.9-1 and the gap acceptance times (t_g) from Figure 28.9M to determine the applicable intersection sight distances for the left-turning vehicle. Where the crossing vehicle must cross more than one lane, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane in excess of one. Where medians are present, the designer will need to consider their effect in the same manner as discussed in Section 28.9.2.2. Figure 28.9N provides the ISD values for all design vehicles and two common left-turning situations.

28.9.6 Measures to Improve Intersection Sight Distance

The available ISD should be checked using the above noted parameters. If the ISD values from the above Sections are provided, no further investigation is needed. If the line of sight is restricted by either bridge railing, guardrail, other obstructions, or the horizontal and vertical alignment of the main road and the ISD value is not available, evaluate one or more of the following modifications, or a combination, to achieve the intersection sight distance:

1. remove the obstructions that are restricting the sight distance,
2. relocate the intersecting road farther from the end of the bridge,
3. widen the structure on the side where the railing is restricting the line of sight,
4. flare the approach guardrail,
5. revise the grades on the main road and/or the intersecting road,
6. close the intersecting road,
7. make the intersecting road one-way away from the main road, and/or
8. review other measures that may be practical at a particular location.

Design Vehicle	Gap Acceptance Time (t_g)(sec)
Passenger Car	5.5
Single-Unit Truck	6.5
Combination Truck	7.5

GAP ACCEPTANCE TIMES
(Left-Turning Vehicles From Major Road)

Figure 28.9M

US Customary						
Design Speed (V_{major}) (mph)	ISD (ft)					
	Passenger Cars		Single-Unit Trucks		Tractor/Semitrailers	
	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes
20	165	180	195	215	225	245
25	205	225	240	265	280	305
30	245	265	290	320	335	365
35	285	310	335	370	390	425
40	325	355	390	425	445	485
45	365	400	430	480	500	545
50	405	445	480	530	555	605
55	445	490	530	585	610	665
60	490	530	575	640	665	725
65	530	575	625	690	720	785
70	570	620	670	745	775	845

Metric						
Design Speed (V_{major}) (km/h)	ISD (m)					
	Passenger Cars		Single-Unit Trucks		Tractor/Semitrailers	
	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes
30	50	50	55	60	65	70
40	65	70	75	80	85	95
50	80	85	95	100	105	115
60	95	100	110	120	130	140
70	110	120	130	145	150	160
80	125	135	145	165	170	185
90	140	155	165	185	190	210
100	155	170	185	205	210	230
110	170	185	200	225	230	255

INTERSECTION SIGHT DISTANCES
(Left-Turning Vehicles from Major Road)

Figure 28.9N

28.10 APPROACHES

The designer is referred to the Department's Approach Standards for Montana Highways for the Department's criteria on approaches. This publication has been prepared by the Department's Traffic Engineering Section in conjunction with the Right-of-Way Bureau and the Maintenance Division.

These regulations are adopted and issued according to the authority granted to the Montana Transportation Commission and/or the Department of Transportation under current Montana Law. Unless otherwise provided or agreed to, they apply to all highways on the Federal-aid system. The frequency, proper placement and construction of points of access to highways are critical to the safety and capacity of those highways. These regulations are intended to provide for reasonable and safe access to highways while preserving their safety and utility to the maximum extent practical. These regulations are not intended to alter or reduce existing or future access control or access limitations, nor are they intended to alter or supersede access, which has been agreed to by appropriate written contract with the Department of Transportation.

28.11 ROUNDABOUTS

28.11.1 General

A roundabout is a channelized intersection where traffic moves around a center island counterclockwise. Modern roundabouts are defined by two basic operational and design principles:

1. Yield-at-Entry. Also known as off-side priority or the yield-to-left rule, yield-at-entry requires that vehicles on the circulatory roadway of the roundabout have the right-of-way and all entering vehicles on the approaches have to wait for a gap in the circulating flow. To maintain free flow and high capacity, yield signs are used as the entry control. As opposed to nonconforming traffic circles, modern roundabouts are not designed for weaving maneuvers, thus permitting small diameters. Even for multilane roundabouts, weaving maneuvers are not considered a design or capacity criterion.

2. Deflection of Entering Traffic. Entrance roadways that intersect the roundabout along a tangent to the circulatory roadway are not permitted. Instead, entering traffic is deflected to the right by the central island of the roundabout and by channelization at the entrance into an appropriate curved path along the circulated roadway. Consequently, no traffic is permitted to follow a straight path through the roundabout.

28.11.2 Design

To provide for increased capacity, modern roundabouts often incorporate flares at the entry by adding lanes before the yield line and have wide circulatory roadways.

Modern roundabouts range in size from mini-roundabouts with inscribed circle diameters as small as 50 ft (15 m), to compact roundabouts with inscribed circle diameters between 100 ft to 115 ft (30 m to 35 m), to large roundabouts, often with multilane circulating roadways and more than four entries up to 500 ft (150 m) in diameter. The greater speeds permitted by larger roundabouts, with inscribed circle diameters greater than 250 ft (75 m) may reduce their safety benefits to some degree.

Designing the geometry of a roundabout involves choosing the best operational and capacity performance while retaining the best safety enhancements. Roundabouts operate most safely when their geometry forces traffic to enter and circulate at slow speeds. Horizontal curvature and narrow pavement widths are used to produce this reduced-speed environment. However, the capacity of roundabouts is negatively affected by these low-speed design elements. As the widths and radii of the entry and circulatory roadways are reduced, the capacity of the roundabout is also reduced. Furthermore, many of the geometric criteria used in design of roundabouts are governed by the design vehicle. Designing a roundabout is a process of determining the optimal balance between safety provisions, operational performance and accommodation of over-sized vehicles.

For additional guidance on the objectives and design of roundabouts, see AASHTO's A Policy on Geometric Design of Highways and Streets, FHWA Publication No. FHWA-RD-00-067, Roundabouts: An Informational Guide and Roundabout Design Guidelines, published by Ourston Roundabout Engineering.