Appendix F

Sight Distance

F.1 SIGHT DISTANCE OVERVIEW

Designing a roadway with adequate sight distance allows vehicles to travel safely and efficiently and perform necessary driving maneuvers. Chapter 2, Section 2.8 provides an overview of the various types of sight distance evaluated in road design. Appendix F provides additional detailed equations and Appendix K provides examples for calculating sight distance. The following types of sight distance will be discussed in this appendix:

1. Stopping Sight Distance
   a. Horizontal Sight Distance
   b. Vertical Sight Distance
2. Intersection Sight Distance
3. Passing Sight Distance
4. Decision Sight Distance

The design team should review the respective section in Chapter 2 and appendix material together to obtain an understanding of the overall approach for evaluating sight distance based on the project context.

F.2 STOPPING SIGHT DISTANCE

This section supplements information regarding stopping sight distance (SSD) provided in Chapter 2, Section 2.8.1.

F.2.1 Horizontal Stopping Sight Distance

This section supplements information regarding horizontal stopping sight distance (SSD) provided in Chapter 2, Section 2.8.1.1.

The needed clearance on the inside of the horizontal curve is calculated using Equation F.2-1 and is illustrated in Exhibit F-1:

$$M = R\left(1 - \cos\left(\frac{90° \times S}{\pi \times R}\right)\right) \qquad \textbf{Equation F.2-1}$$

where:

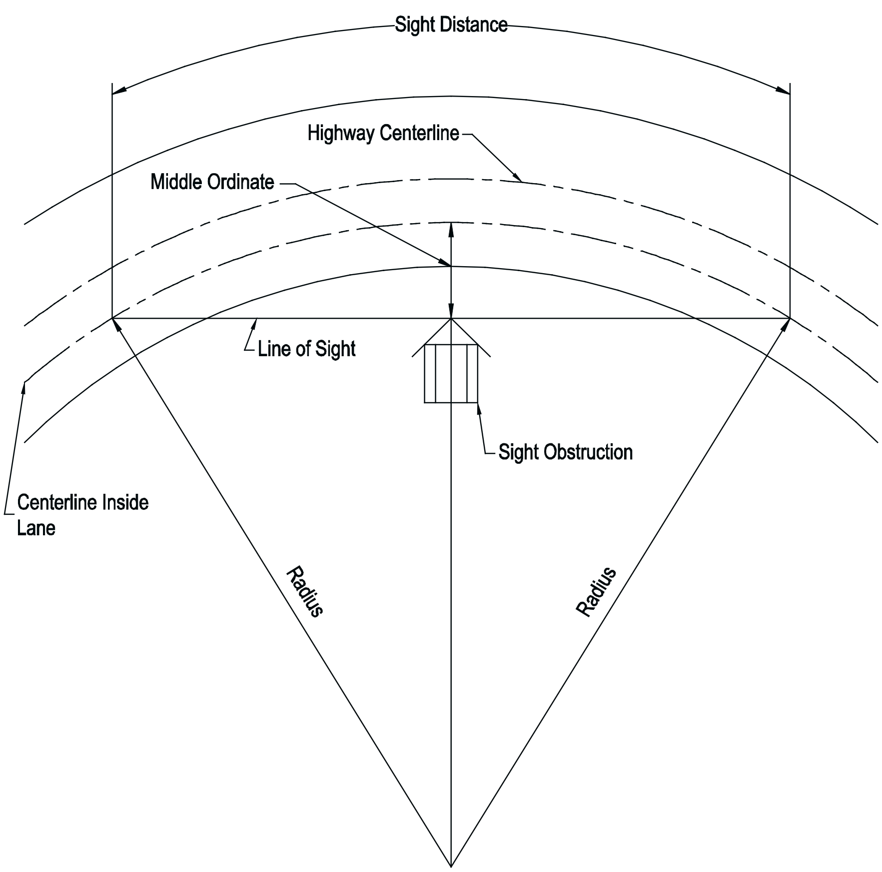
M = middle ordinate, or distance from the center of the inside travel lane to the obstruction, ft

R = radius of curve, ft

S = stopping sight distance, ft

Note: The expression $\left(\frac{90° \times S}{\pi \times R}\right)$ is in degrees, not radians.

Exhibit F-1
Horizontal Stopping Sight Distance



The M values as calculated using Equation F.2-1 apply between the Point of Curvature (PC) and Point of Tangency (PT) of a horizontal curve (or from the spiral to curve (SC) to the curve to spiral (CS)). In addition, some transition is needed on the entering and exiting portions of the curve. The design team should typically use the following steps:

1. Locate the point which is on the edge of travel lane and a distance of $S/2$ before the PC or SC.
2. Locate the point which is a distance M measured laterally from the center of the travel lane at the PC or SC.
3. Connect the two points located in Step 1 and 2. The area between this line and the roadway should be clear of all continuous sight obstructions.

4. A symmetrical application of Step 1 through 3 should be used beyond the PT or CS.

F.2.2 Vertical Alignment Sight Distance Considerations

When developing vertical alignments, the equations for determining the length of crest vertical curves that provide the desired sight distance are generally adequate to ensure a profile design meets the sight distance criteria. However, it is important to recognize that these equations are based on the geometry of a single parabolic curve on straight horizontal alignment, and may not provide a true representation of actual sight distance when shorter curves (relative to sight distance) are closely spaced. Similarly, roadside features where horizontal and vertical curves are used together can influence sight distance beyond what can be determined easily with equations. For these instances, checking sight distance graphically may be appropriate.

To better understand the limitations of the crest curve equations, it is helpful to understand when and how they are applied. The most familiar Equation F.2-2, is used when the length of the curve is equal to or greater than the sight distance needed. When S is less than L,

$$L = KA$$

Equation F.2-2

where:

L = length of vertical curve, feet

K = horizontal distance (feet) needed to produce a 1-percent change in gradient

A = algebraic difference between the two tangent grades, percent

$$K = \frac{S^2}{200(\sqrt{h_1} + \sqrt{h_2})^2}$$

Equation F.2-3

S = sight distance, feet

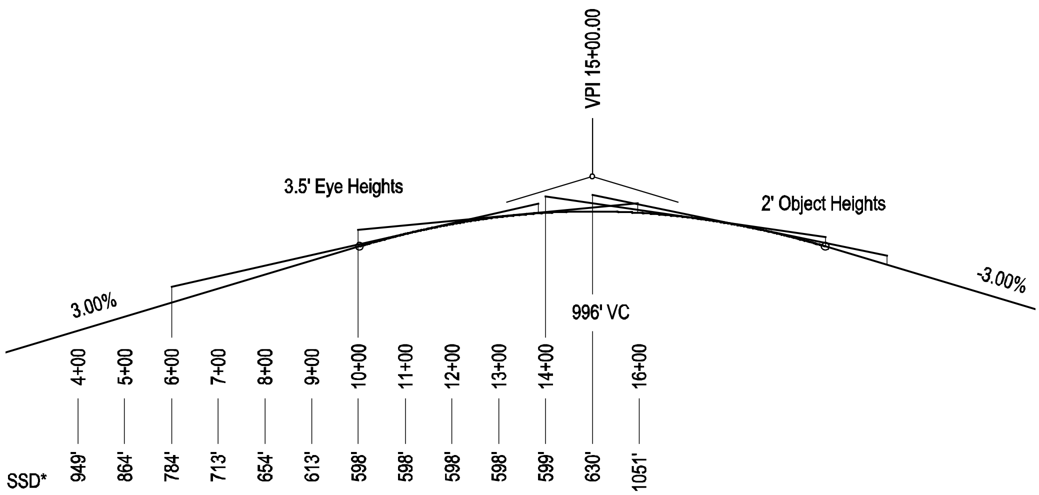
h_1 = height of eye above road surface, feet

h_2 = height of object above road surface, feet

Equation F.2-2 is often used for all crest curve lengths, and provides a conservative length requirement for smaller values of A. Unless influenced by a horizontal curve, sight distance is provided if this equation is satisfied for any length crest curve. When the sight distance provided by the curve is shorter than its length, the sight distance reduces to a minimum as a vehicle (eye) approaches the curve. The minimum sight distance is provided when both the eye and object are on the vertical curve, and begins to increase once the object reaches the tangent grade.

Exhibit F-2 illustrates stopping sight distance for a passenger car traveling over the minimum length curve for a 60 mph design speed, given a 6-percent algebraic difference in grades and adjusted for 3 percent downgrades. Stopping sight distance is shown at even stations, for a vehicle traveling left to right.

Exhibit F-2 Stopping Sight Distance over Minimum Length of Curve



*Horizontal Distance, Sloped Sight Line May Be Used But Must Be Adjusted For 10:1 Scale Shown

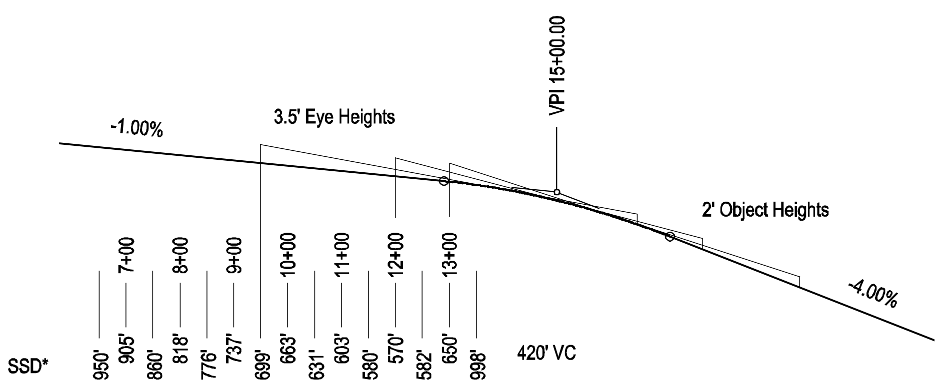
When S is greater than L,

Equation F.2-4

$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A}$$

When the sight distance provided by the curve exceeds its length, the minimum sight distance is provided at one point, when the vertical curve is between the eye and the object, and each is on a tangent grade. The distance between the eye and curve and the object and curve when sight distance is a minimum depends on their heights. For passing sight distance, the heights are the same and the minimum sight distance occurs when the eye and object are equidistant from the curve. For stopping sight distance, this point occurs when the eye is about twice as far from the start of the curve as the object is from the end of the curve. Exhibit F-3 illustrates stopping sight distance at 50-foot intervals for a passenger car traveling from left to right.

Exhibit F-3 Stopping Sight Distance for Passenger Car



*Horizontal Distance, Sloped Sight Line May Be Used But Must Be Adjusted For 10:1 Scale Shown

Using Equation F.2-4, the minimum SSD is found to be about 570 feet, corresponding to a 60 mph design speed for level grades or 55 mph for the 4-percent down grade encountered for the direction shown. Assuming the full 4-

percent down grade through the breaking distance, stopping sight distance for 60 mph is 610 feet, which is provided except for about 200 feet between stations 10+50 and 13+00. Plotting sight lines graphically can show where sight distance is reduced, and can help the design team locate or check the sight distance at decision points. For symmetrical vertical curves, the sight distance in either direction is equal for points that are the same distance from the vertical point of intersection. In this case, the minimum stopping sight distance for a vehicle traveling back on stationing would be 570 feet at about station 18+00.

Rather than calculating a curve length using Equation F.2-4 and checking it against the sight distance to determine which equation is appropriate, check the threshold algebraic difference, shown as A' below, to determine which equation applies for a specific sight distance for intersecting grades. To determine this threshold value for a given sight distance and K-value, set L equal to S and solve for A:

$$A' = \frac{S}{K}$$

Equation F.2-5

Exhibit F-4 represents these threshold values for stopping sight on crest curves with level grades. Similar tables can be produced for sag curves, or for passing sight, decision sight, and any sight distance adjusted for grades.

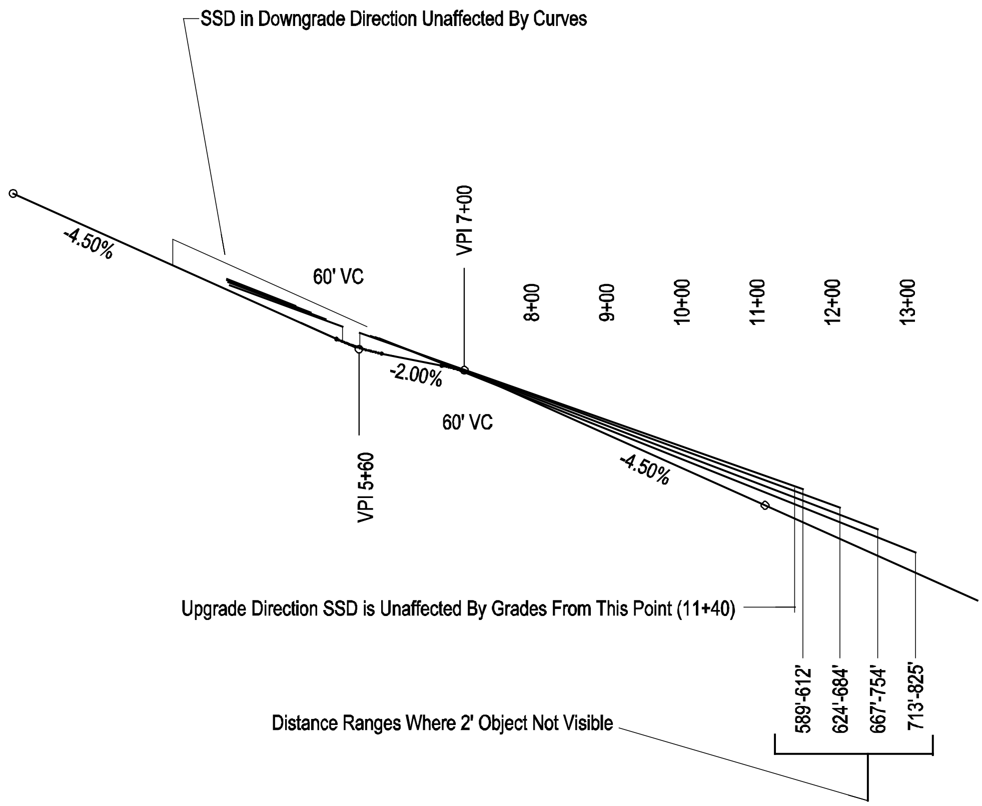
Design Speed (mph)	SSD Level (feet)	K (feet/1% change in grade)	A' (%)
15	80	3.0	26.98
20	115	6.1	18.77
25	155	11.1	13.92
30	200	18.5	10.79
35	250	29.0	8.63
40	305	43.1	7.08
45	360	60.1	5.99
50	425	83.7	5.08
55	495	113.5	4.36
60	570	150.6	3.79
65	645	192.8	3.35
70	730	246.9	2.96
75	820	311.6	2.63
80	910	383.7	2.37

Exhibit F-4 Stopping Sight Distance on Crest Curves with Level Grades

For algebraic differences greater than A', Equation F.2-2 should be used to determine curve length. Note that for very slow speeds, vertical sight distance is generally not a critical factor and the minimum length equation L=3V results in a longer length than either SSD equation for most values of A.

When shorter vertical curves are closely spaced, the standard equations may not be adequate to check available sight distance. These equations are based on the geometry of a parabola bounded by tangent grades that extend indefinitely, and cannot account for the effect that adjacent curves may have on the elevation of the eye or object. The most common application of shorter curves relative to sight distance is in urban environments where very short curves may be necessary to fit within vertical controls and for longer sight distances such as passing sight distance, decision sight distance, or sight distances adjusted for steep downgrades. Exhibit F-5 below may represent a case where a road grade is modified to accommodate an existing intersecting roadway.

Exhibit F-5
Stopping Sight Distance at an Intersecting Roadway



Based on Equation F.2-4, the crest curve at station 7+00 has a stopping sight distance of about 462 feet (228 feet based on Equation F.2-2). By checking the alignment graphically, it is shown that the SSD provided is unlimited through the curves for vehicles traveling left to right, and limited only by small gaps beyond 585 feet in the other direction.

F.3 INTERSECTION SIGHT DISTANCE

This section supplements information regarding intersection sight distance (ISD) provided in Chapter 2, Section 2.8.2.

As stated in Chapter 2, MDT uses gap acceptance as its basic methodology in the design of intersection sight distance. Additional information on gap acceptance is provided in the *AASHTO Green Book (2).*

The following sections describe the specific design considerations, criteria and equations for each of the following types of traffic control:

- No Traffic Control (AASHTO Case A)
- Stop Controlled/Traffic Signal Controlled (AASHTO Case B and D)

- Vehicles Entering Major Roadway (AASHTO Case B1 and Case B2)
- Straight Through Crossing Vehicle (AASHTO Case B3)

- Yield Control (AASHTO Case C)
- All-Way-Stop (AASHTO Case E)
- Stopped Vehicle Turning Left (AASHTO Case F)
- Channelized Right-Turn
- Roundabouts

F.3.1 No Traffic Control (AASHTO Case A)

As stated in Section 2.8.2, 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 crash, which is typically 50-percent of their mid-block running speed. Exhibit F-6 illustrates the ISD and sight lines between vehicles. Exhibit F-7 provides the ISD criteria for intersections with no traffic control. For approach grades greater than 3-percent, adjust the ISD values obtained in Exhibit F-7 with the applicable ratios in Exhibit F-8.

Exhibit F-6
Intersection Sight Distance Components
(No Traffic Control)

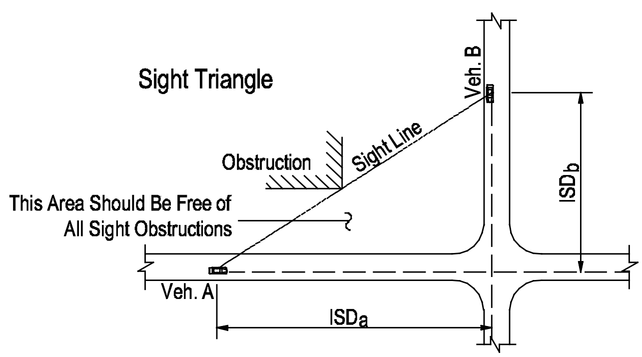


Exhibit F-7
Intersection Sight Distance Criteria
(No Traffic Control)

Design Speed (mph)	Intersection Sight Distance (ft)
15	70
20	90
25	115
30	140
35	165
40	195
45	220
50	245

Note: For approach grades greater than 3-percent, multiply the sight distance values in this table by the appropriate adjustment factor from Exhibit F-8. The grade adjustment is based on the approach roadway grade only.

Exhibit F-8
Approach Factors for Approach Sight Distance Based on Approach Grade (No Traffic Control)

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	0.9	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

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.

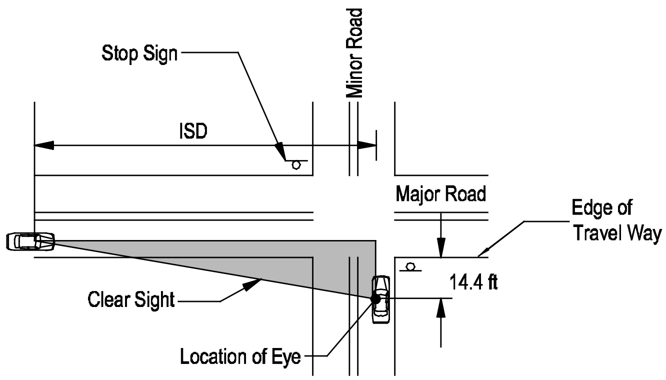
F.3.2 Stop Controlled/Traffic Signal Controlled (AASHTO Case B and Case D)

As stated in Chapter 2, Section 2.8.2, 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.

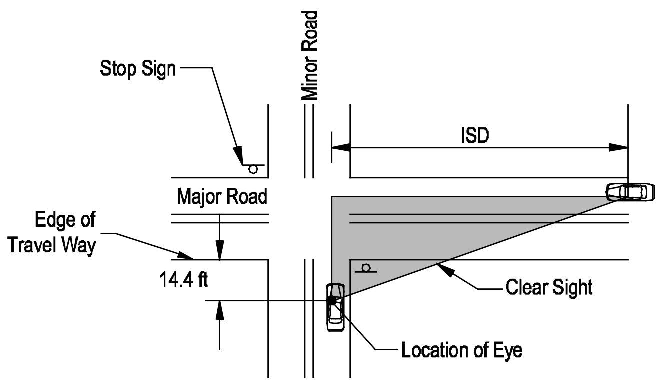
If a signalized intersection implements two-way flashing operations or right-turns are permitted on red, the stop-controlled criteria may apply for intersection sight distance.

MDT 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 Exhibit F-9 (refer to Chapter 2, Exhibit 2-8).

Exhibit F-9
Clear Sight Triangles for Stop Controlled Intersections



Clear Sight Triangle For Viewing



Clear Sight Triangle For Viewing

The lengths of the legs of these sight triangles are determined as follows:

1. **Minor Road**. The length of the 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 14.4 feet from the edge of traveled way (excluding shoulder and bicycle lanes) for the major road and in the center of the lane on the minor road; see Exhibit F-9. 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 for vehicles approaching 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 Exhibit F-9.
2. **Major Road**. The length of the sight triangle leg or ISD along the major road is determined using Equation F.3-1:

Equation F.3-1

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

where:

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

V_{major} = design speed of major road, mph

t_g = gap acceptance time for entering the major road, s

The gap acceptance time (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.

Within this clear sight triangle, if practical, remove, lower any object or trim lower branches that obstruct the driver's view to 3.5 feet or below. These objects may include buildings, parked or turning vehicles, trees, hedges, tall crops, un-mowed grass, fences, retaining walls and the existing ground line. In addition, where an interchange ramp intersects the major road or crossroad near a bridge on a crest vertical curve, objects such as bridge parapets, piers, abutments or the crest vertical curve itself may restrict the clear sight triangle.

F.3.2.1 Vehicle Entering Major Roadway (AASHTO Case B1 and Case B2)

To determine the intersection sight distance for vehicles turning left or right onto the major road, the design team should use Equation F.3-1 and the gap acceptance time (t_g) presented in Exhibit F-10. Exhibit F-11, which solves Equation F.3-1, provides the ISD values for all design vehicles turning left on two-lane, level facilities. Exhibit F-12, which solves Equation F.3-1, provides the ISD values for all design vehicles turning right on two-lane, level facilities.

Exhibit F-10
Gap Acceptance Times for Right and Left Turn from a Minor Road

Design Vehicle	Left-Turn from Stop, Gap Acceptance Time (t_g) (s)	Right-Turn from Stop, Gap Acceptance Time (t_g) (s)
Passenger Car	7.5	6.5
Single-Unit Truck	9.5	8.5
Tractor/Semitrailer	11.5	10.5

Exhibit F-11
Two-Lane Intersection Sight Distances for Left-Turn from a Minor Road

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

Exhibit F-12
Two-Lane Intersection Sight Distances for Right-Turn from a Minor Road

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	1085

The design team should also consider the following:

1. **Multilane Facilities.** For multilane facilities, the gap acceptance times presented in Exhibit F-10 should be adjusted to account for the additional distance required by the turning vehicle to cross the additional lanes or median. The following will apply:
    a. **Left-Turns.** 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.
    b. **Right-Turns**. Because the turning vehicle is assumed to be turning into the nearest right through lane, no adjustments to the gap times are required.
2. **Medians.** For a multilane facility which does not have a median wide enough to store a stopped vehicle, divide the median width by 12 feet to determine the corresponding number of lanes, and then use the criteria in Item 1a above to determine the appropriate time factor. On multilane facilities with a median wide enough to store the stopped vehicle, the design team should evaluate the move in two steps; see Exhibit F-13:
    a. First, with the vehicle stopped on the minor road (the bottom portion in Exhibit F-13), use the gap acceptance times and distances for a vehicle turning right (Exhibit F-10 and Exhibit F-12) to determine the applicable ISD. Under some circumstances, it may be necessary to check the crossing maneuver to determine if it is the critical movement. Crossing criteria are further discussed in the intersection design material presented in Chapter 6 and Section F.3.2.2 below.
    b. Then, with the vehicle stopped in the median (top portion in Exhibit F-13), assume a two-lane roadway design and use the gap acceptance times and distances for vehicles turning left (Exhibit F-10 and Exhibit F-11) to determine the applicable ISD.

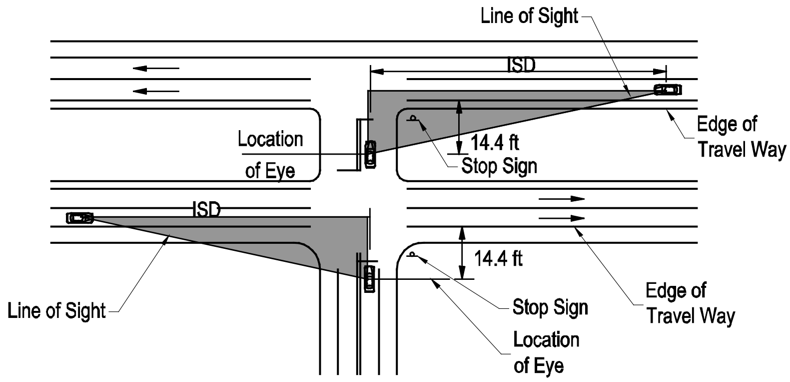


Exhibit F-13 Intersection Sight Distance for Divided Facilities

3. **Approach Grades.** If the approach grade on the minor road exceeds +3 percent upgrade, add the following times to the basic gap acceptance times in Exhibit F-10:
   a. **Left-Turns.** Multiply the percent grade on the approach by 0.2 and add this to the base time gap. This does not apply if the approach grade is negative.
   b. **Right-Turns.** Multiply the percent grade on the approach by 0.1 and add this to the base time gap. Use the adjusted t_g in Equation F.3-1 to determine the applicable ISD. Do not apply the grade adjustment if the approach grade is negative.
4. **Trucks.** At intersections near truck stops, interchange ramps, and grain elevators, the design team should consider using 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 Exhibit F-10. ISD values for level, two-lane roadways are presented in Exhibit F-11 and Exhibit F-12.
5. **Height of Eye/Object.** The height of eye for passenger cars is assumed to be 3.5 feet above the surface of the minor road. The height of object (approaching vehicle on the major road) is also assumed to be 3.5 feet. An object height of 3.5 feet 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 are enough trucks to warrant their consideration, assume an eye height of 7.9 feet for a tractor/semitrailer and 5.9 feet for single-unit trucks and buses. If a truck is the assumed entering vehicle, the object height will still be 3.5 feet for the passenger car on the major road.
6. **Skew.** At skewed intersections where the intersection angle is less than 60 degrees, 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 Item 1 above and/or Section F.3.2.2, determine the appropriate ISD value based on this extra travel distance.

F.3.2.2 Straight Through Crossing Vehicle (AASHTO Case B3)

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.

Use Equation F.3-1 and the gap acceptance times (t_g) from Exhibit F-14 and the adjustment factors to determine the ISD for crossing maneuvers. Where medians are present, include the median width in the overall length to determine the applicable gap time. Divide this width by 12 feet to determine the corresponding number of lanes for the crossing maneuver.

Exhibit F-14
Gap Acceptance Times for Crossing Maneuvers on Two-Lane Facilities

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

The following adjustments can be made to Exhibit F-14.

1. **Multilane Highway**. 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.
2. **Approach Grade.** If the approach grade on the minor road exceeds +3 percent upgrade, multiply the percent grade of the minor road approach by 0.2 and add it to the base gap acceptance time. Do not apply the grade adjustment if the approach grade is negative.

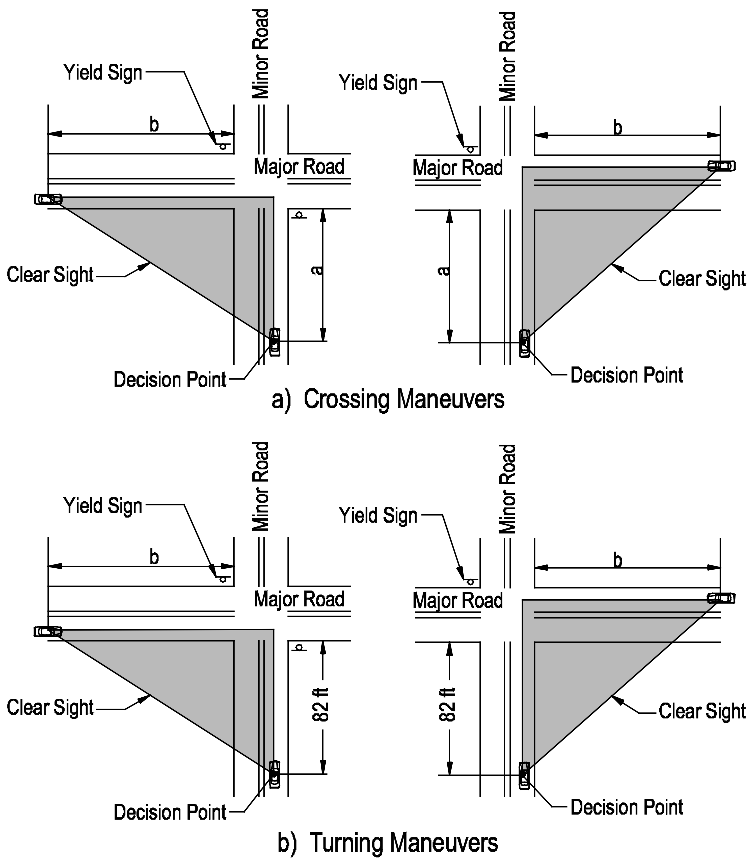
F.3.3 Yield Control (AASHTO Case C)

At intersections controlled by a yield sign (except roundabouts, which are described in Section F.3.7), drivers on the minor road will typically:

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

Yield control criteria are based on a combination of the no traffic control ISD discussed in Section F.3.1 and the stop-controlled ISD as discussed in Section F.3.2. To determine the applicable clear sight triangles of the approaches, the following will apply; see Exhibit F-15:

Exhibit F-15 Intersection Sight Distance for Yield Control



1. **Crossing Maneuver**. Use the following to determine the legs of the clear sight triangle; see Exhibit F-15:
   a. **Minor Road.** The leg on the minor road approach can be determined directly from Exhibit F-16.
   b. **Major Road.** The leg on the major road is determined using Equation F.3-2 and the times listed in Exhibit F-16:

Equation F.3-2

$$t_g = t_a + \frac{w + L_a}{0.88(V_{minor})}$$

$$b = 1.47(V_{major})(t_g)$$

where:

b = length of leg of sight triangle along the major road, ft

t_g = travel time to reach and clear the major road in a crossing maneuver, s

t_a = travel time to reach the major road from the decision point for a vehicle that does not stop, s (use appropriate value for the minor-road design speed from Exhibit F-16, adjusted for approach grade, where appropriate)

w = width of intersection to be crossed, ft

L_a = length of design vehicle, ft

V_{minor} = design speed of minor road, mph

V_{major} = design speed of major road, mph

Exhibit F-16
ISD Assumptions for Yield Control Intersections Crossing Maneuvers

Major Road Design Speed (mph)	Approach Distance Along Minor Road (1) (*a*) (ft)	Travel Time From Decision Point to Major Road (t_a) (1) (2) (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

(1) For minor-road approach grades that exceed +3 percent upgrade, multiply by the appropriate adjustment factor from Exhibit F-8. 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.

2. **Turning Maneuvers.** For the turning left or right vehicle, the approach legs are determined as follows; see Exhibit F-15:
   a. **Minor Road.** The assumed turning speed from the minor road to the major road is 10 mph. This corresponds to an approach distance of 82 feet along the minor road leg.
   b. **Major Road.** To determine the legs along the major road, use the same procedures as discussed in Section F.3.2 for the stop controlled intersection, Equation F.3-1 and the gap acceptance time listed in

Exhibit F-17. Because the gap acceptance time is longer than the stop-controlled gap time, it will be unnecessary to determine the sight distance criteria for the vehicle which stops at the yield sign.

Exhibit F-17
Gap Acceptance Times for Turning Maneuvers at Yield Control Intersections

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

If the approach grade on the minor road exceeds +3 percent upgrade, 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. Do not apply the grade adjustment if the approach grade is negative.
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. Do not apply the grade adjustment if the approach grade is negative.

F.3.4 All-Way Stop (AASHTO Case E)

No additional information is provided in this appendix associated with this type of traffic control.

F.3.5 Stopped Vehicle Turning Left from Major Road (AASHTO Case F)

As stated in Section 2.8.2, at all intersections, regardless of the type of traffic control, the design team should consider the sight distance needs for a stopped vehicle turning left from the major road. An illustration of this situation is shown in Exhibit F-18. 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 will only 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 the intersection.

Use Equation F.3-1 and the gap acceptance times (t_g) from Exhibit F-19 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 design team will need to consider their effect in the same manner as discussed in Section F.3.2. Exhibit F-20 provides the ISD values for all design vehicles and two common left-turning situations.

Exhibit F-18
Diagram for Intersection Sight Distance for a Stopping Vehicle Turning Left on a Major Road

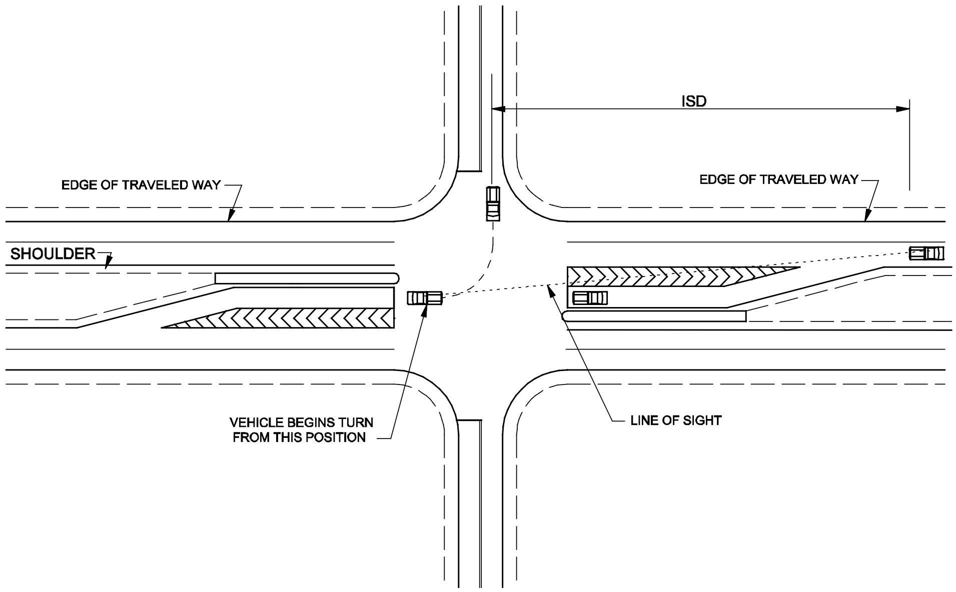


Exhibit F-19
Gap Acceptance Times for Left-Turning Vehicle on a Major Road

Design Vehicle	Gap Acceptance Time (t_g) (sec)
Passenger Car	5.5
Single-Unit Truck	6.5
Tractor/Semi-Trailer	7.5

Exhibit F-20 Intersection Sight Distances for Left-Turning Vehicles from a Major Road

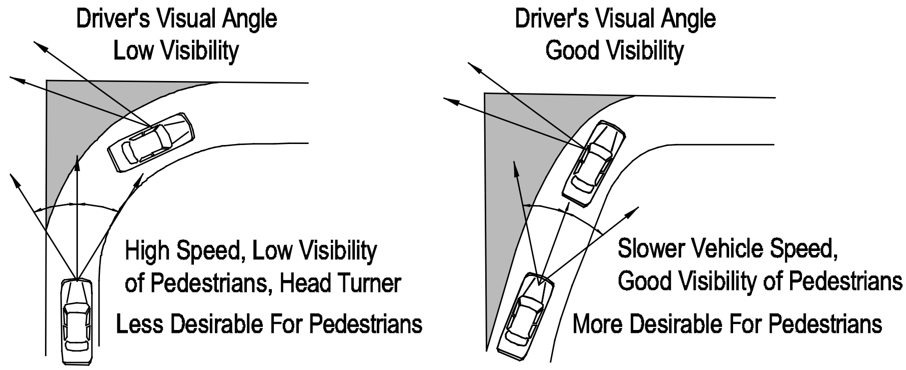
Design Speed (V_{major}) (mph)	ISD (ft)					
	Passenger Cars		Single-Unit Trucks		Tractor/Semitrailers	
	Cross 1 lane	Cross 2 lanes	Cross 1 lane	Cross 2 lanes	Cross 1 lane	Cross 2 lanes
20	165	180	195	215	225	245
25	205	225	240	265	280	305
30	245	265	290	320	335	365
35	285	310	335	375	390	425
40	325	355	385	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
75	610	665	720	795	830	905
80	650	710	765	850	885	965

F.3.6 Channelized Right-Turn

Sight lines at a channelized right-turn should be clear of obstructions and provide sufficient visibility for various users.

As stated in Section 2.8.2, when designing a channelized right-turn lane at an intersection, the sight distance for the approaching vehicles and sight distances for the pedestrians approaching the intersection should be considered. Sight lines should be clear of obstructions and provide sufficient visibility for various users. Exhibit F-21 illustrates two different designs for a channelized right-turn and the visibility from each design *(4)*. The image on the left shows a circular curve where motorists in the channelized right-turn have an abrupt angle (looking over their shoulder) to identify a gap in the oncoming traffic. The image on the right shows an arrangement of compound curves to slow vehicles in the vicinity of the crosswalk, as well as provide an appropriate angle for sight distance when turning right onto the cross street.

Exhibit F-21
Intersection Sight Distance for a Channelized Right-turn

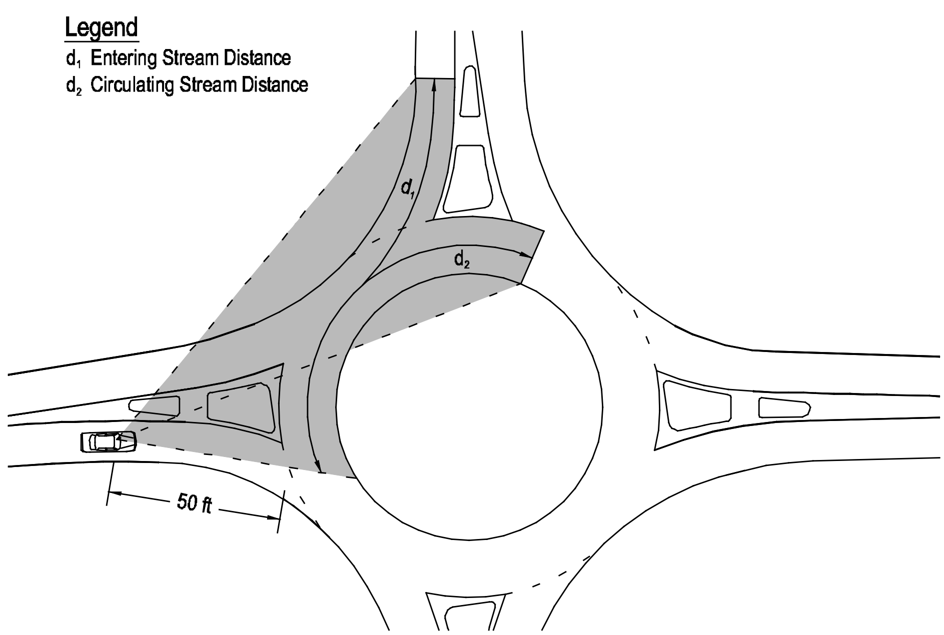


F.3.7 Roundabouts

A stated in Chapter 2, Section 2.8.2, intersection sight distance should be evaluated at the entries of a roundabout. At roundabouts, the sight triangle should follow the curvature of the roadway, and thus distances should be measured not as straight lines but as distances along the vehicular path. *NCHRP Report 672: Roundabouts An Information Guide, Second Edition* describes the method for evaluating intersection sight distance at roundabouts, which is described below *(3)*.

Exhibit F-22 (refer to Chapter 2, Exhibit 2-9) presents a diagram showing the method for determining intersection sight distance.

Exhibit F-22
Intersection Sight Distance at Roundabouts



As shown in Exhibit F-21, there are two conflicting approaches on the sight triangle that should be evaluated, which are further described below.

1. **Length of Approach Leg.** The length of the approach leg of the sight triangle should be limited to 50 feet. This distance encourages vehicles to slow down prior to entering the roundabout, which supports the need to slow down and yield at the roundabout entry. This also allows drivers to identify any potential pedestrians crossing in advance of the roundabout

entry. If the approach leg of the sight triangle is greater than 50 feet, landscaping can be added to restrict sight distance to the minimum requirements *(3)*.

2. **Length of Conflicting Leg.** A vehicle approaching the entry of a roundabout may encounter conflicting vehicles within the circulatory roadway of the roundabout and vehicles entering the roundabout from the immediate upstream entry. The length of the conflicting leg is calculated using Equations F.3-3 and F.3-4.

$$d_1 = 1.47\left(V_{major,\ entering}\right)(t_c)$$

Equation F.3-3

$$d_2 = 1.47(V_{major,\ circulating})(t_c)$$

Equation F.3-4

Where:

d_1 = length of the entering leg of sight triangle, ft

d_2 = length of the circulating leg of sight triangle, ft

V_{major} = design speed of conflicting movement, mph

t_c = critical headway for entering the major road, s, equal to 5.0s.

The design speed of the conflicting movements ($V_{major,\ entering}$ and $V_{major,\ circulating}$) is estimated using the following. Additional information regarding the speeds described below (R_1, R_2, and R_4) is illustrated in Chapter 6, Exhibit 6-13 and further described in *NCHRP Report 672 (3).*

Additional information regarding the speeds described below (*R_1*, *R_2*, and *R_4*) is illustrated in Chapter 6, Exhibit 6-13 and further described in *NCHRP Report 672 (3).*

1. **Entering stream.** This consists of the vehicles from the immediate upstream entry. The speed for this movement can be approximated by taking the average of the theoretical entering (R_1) speed and the circulating (R_2) speed.
2. **Circulating stream.** This consists of the vehicles that enter the roundabout prior to the immediate upstream entry. This speed can be approximated by taking the speed of left-turning vehicles (path with radius R_4).

The critical headway for entering the major road (t_c) is based on the amount of time required for a vehicle to safely enter the conflicting stream. This value is typically 5.0 seconds, which is based on the critical headway required for passenger cars. Exhibit F-23 summarizes the length of conflicting leg for various approach speeds of an intersection sight triangle *(3)*.

Exhibit F-23 Conflicting Leg for Sight Triangle at Roundabouts

Conflicting Approach Speed (mph)	Distance (ft)
10	73.4
15	110.1
20	146.8
25	183.5
30	220.2

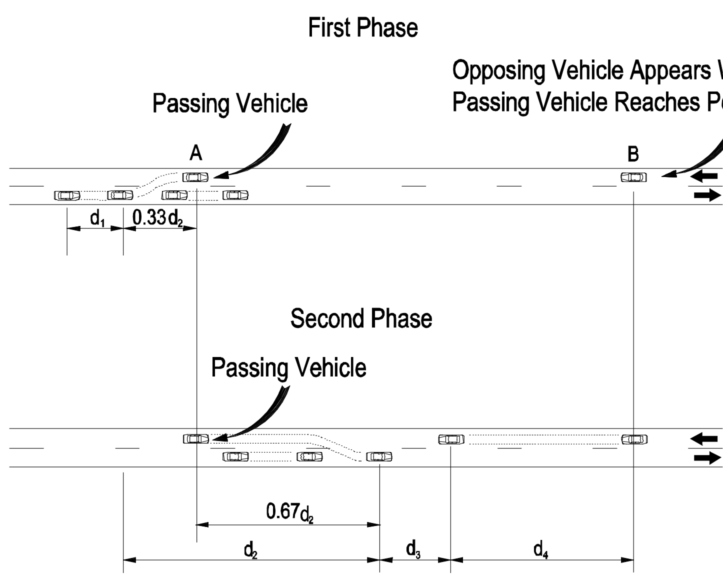
After the stopping sight distance (discussed in Section F.2) and the intersection sight distance at a roundabout have been evaluated separately, the sight triangles should be overlaid onto a single drawing to allow for an overall combined check for each approach. This can help provide design guidance for landscaping design and other treatments. Additional information regarding roundabout design is provided in the intersection design material presented in Chapter 6.

F.4 PASSING SIGHT DISTANCE

This section supplements information regarding passing sight distance (PSD) provided in Chapter 2, Section 2.8.3.

The minimum passing sight distance for two-lane highways is determined from the sum of four distances as illustrated in Exhibit F-24 (refer to Chapter 2, Exhibit 2-10).

Exhibit F-24
Elements of Passing Distance
(Two-Lane Highways)



The following discussion provides the basic assumptions used to develop passing sight distance values for design.

1. **Initial Maneuver Distance (*d_1*).** As stated in Chapter 2, Section 2.8.3, this is the distance traveled during the perception and reaction time and during the initial acceleration to the point of encroachment on the left lane. For the initial maneuver, the overtaken vehicle is assumed to be traveling at a uniform speed.

   The average speed of the passing vehicle is assumed to be approximately 9 mph greater than the overtaken vehicle. Use Equation F.4-1 to determine d_1:

$$d_1 = 1.47t_1\left(v - m + \frac{at_1}{2}\right)$$

Equation F.4-1

The AASHTO Green Book can provide additional information on the variables described *(2)*.

where:

t_1 =	time of initial maneuver, s
v =	average speed of passing vehicle, mph
m =	difference in speed of passed vehicle and passing vehicle, mph
a =	average acceleration, mph/s

2. **Distance of Passing Vehicle in Left Lane (d_2).** As stated in Chapter 2, Section 2.8.3, this is the distance traveled by the passing vehicle while it occupies the left lane. Use Equation F.4-2 to determine d_2:

$$d_2 = 1.47vt_2$$

Equation F.4-2

where:

v =	average speed of passing vehicle, mph
t_2 =	time passing vehicle occupies the left lane, s

3. **Clearance Distance (d_3).** As stated in Chapter 2, Section 2.8.3, this is the distance between the passing vehicle at the end of its maneuver and the opposing vehicle.
4. **Opposing Vehicle Distance (d_4).** As stated in Chapter 2, Section 2.8.3, this is the distance traveled by an opposing vehicle during the time the passing vehicle occupies the left lane. The opposing vehicle appears after approximately one-third of the passing maneuver (d_2) has been accomplished. The opposing vehicle is assumed to be traveling at the same speed as the passing vehicle.

Exhibit F-5 (refer to Chapter 2, Exhibit 2-11) provides the minimum passing sight distance for design on two-lane, two-way highways. The AASHTO Green Book can provide additional information on the variables described *(2)*.

Exhibit F-25
Minimum Passing Sight Distance (Two-Lane Highways)

Design Speed (mph)	Assumed Speeds Passed Vehicle (mph)	Passing Vehicle (mph)	Minimum PSD for Design (ft)
30	18	30	500
35	23	35	550
40	28	40	600
45	33	45	700
50	38	50	800
55	43	55	900
60	48	60	1000
65	53	65	1100
70	58	70	1200
75	63	75	1300
80	68	80	1400

F.5 DECISION SIGHT DISTANCE

This section supplements information regarding decision sight distance (DSD) provided in Chapter 2, Section 2.8.4.

Equation F.5-1 describes the decision sight distance for avoidance maneuvers A and B. Refer to Exhibit 2-12 in Chapter 2 that summarizes the *DSD* for various speeds and maneuvers. For these avoidance maneuvers, the braking distance is added to the pre-maneuver component.

Equation F.5-1

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

where:

DSD = decision sight distance, ft

t = pre-maneuver time, s

V = design speed, mph

a = driver deceleration, ft/s²

Equation F.5-2 describes the decision sight distance for avoidance maneuvers C, D and E. For these avoidance maneuvers, the braking distance is replaced with a maneuver distance based on maneuver times that decrease with increasing speed.

$$DSD = 1.47Vt$$

Equation F.5-2

where:

- DSD = decision sight distance, ft
- t = total pre-maneuver and maneuver time, s
- V = design speed, mph

Additional information on decision sight distance is provided in Chapter 2, Section 2.8.4.

F.6 REFERENCES

1. Montana Department of Transportation (MDT). *Baseline Criteria Practitioner's Guide.* Helena, MT: MDT, 2021.
2. AASHTO. *A Policy on Geometric Design of Highways and Streets.* Washington, D.C.: AASHTO, 2018.
3. Rodegerdts, L., J. Bansen, C. Tiesler, J. Knudsen, E. Myers, M. Johnson, M. Moule, B. Persaud, C. Lyon, S. Hallmark, H. Isebrands, R. B. Crown, B. Guichet, and A. O'Brien. *NCHRP Report 672: Roundabouts: An Informational Guide, 2nd ed.* Transportation Research Board of the National Academies, Washington, D.C., 2010.
4. Harwood, D. and C. Hoban. *Low Cost Methods for Improving Traffic Operations on Two-Lane Roads*, Report No. FHWA-IP-87-2. Washington, D.C.: FHWA, 1987.