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MINNESOTA DEPARTMENT OF TRANSPORTATION	**TRANSMITTAL LETTER NO. (19-03)**
DEVELOPED BY: Design Standards Unit	**MANUAL:** Road Design Manual
ISSUED BY: Office of Project Management and Technical Support	**DATED:** September 17, 2019
SUBJECT: Section 5-2, other minor edits	

A list of changes is attached to this update.

INSTRUCTIONS:

1. Record this transmittal letter number, date and subject on the transmittal record sheet located in the front of the manual. The last Transmittal Letter was 19-02, dated May 29, 2019.

2. Remove from the manual: Section 2-7(1-2)
Section 5-2(1-10)
Section 5-4(5-7)
Section 6-3(3-4)

3. Insert into the manual: Section 2-7(1-2)
Section 5-2(1-10)
Section 5-4(5-7)
Section 6-3(3-4)

4. The Road Design Manual and associated Transmittal Letters are available online in PDF format at. http://roaddesign.dot.state.mn.us/roaddesign.aspx

5. Any technical questions regarding this transmittal should be directed to Mike Elle, Design Standards Engineer, at (651) 366-4622, or by email to DesignStandards.DOT@state.mn.us

Michael Elle

Michael Elle, P.E.
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Summary of Changes
MnDOT Road Design Manual
19-03

General

- Comprehensive replacement of old intersection sight distance (ISD) criteria—previously based on predecessor AASHO policies and a variety of research findings—with current Green Book criteria that are based on NCHRP Report 383 (1996)

Section 5-2 GENERAL INTERSECTION DESIGN

- Revise title from “INTERSECTIONS”
- 5-2.0
    - Remove verbiage on degree of access control
    - Soften verbiage to be realistic about safety and efficiency within space and funding limits
    - Include newer and innovative intersection configurations
- 5-2.01 Design Principles
    - Revise title from “Design Vehicle Selection” and repurpose discussion likewise
    - New paragraph stating high-level objective(s) and their relationship(s) to design and traffic control selections
    - New paragraph to emphasize right-sizing of intersections for safety and operational performance
    - New paragraph clarifying the operational basis of standard ISD and the role of stopping sight distance (SSD) in design decision making
- 5-2.02 Intersection Sight Distance
    - Remove opening sentence to avoid portraying ISD as black and white
    - Add verbiage from the PBPD guidance document discussing potential sight distance improvements as safety remedies
        - Replaces verbiage stating that practically any improvements would be cost justified
    - Incorporate verbiage from the PBPD guidance document pertaining to the role of SSD in intersection safety design and recommending a performance-based design thought process to evaluate the need for SSD correction
    - New paragraph referencing safety / sight distance relationships provided in NCHRP Report 875 with a short discussion pertaining to general approach
    - New paragraph with thumbnail sketch of fundamental ISD framework and principles
- 5-2.02.01 Geometric Framework
    - Per new research and Green Book bases, establish sight triangles as the governing framework for all control cases (as opposed to the stand-alone discussion in previous editions)
    - Remove safety and comfort content, which is now discussed in previous sections
    - Add additional items to the list of things that may restrict sight lines
    - Add figures and discussion paraphrased from the Green Book and/or NCHRP Report 383 pertaining to approach and departure sight triangles
        - Remove superseded Road Design Manual content
- 5-2.02.02 Intersection Control Cases
    - Revise title from “Policy”
    - Add general discussion on application of sight triangles to the range of intersection controls
    - Re-numbering, reorganization and re-titling of all subsections to conform to current Green Book intersection control cases
    - Complete removal and replacement of previous Road Design Manual content for uncontrolled, yield-controlled, and stop-controlled cases with new content from the Green Book and/or NCHRP Report 383
- 5-2.02.02.04 Case D—Intersections with Traffic-Signal Control
    - Revise title from its previous case classification and name
    - Besides the complete removal and replacement described above, retain verbiage requiring that drivers must be able to see the signal control device
- 5-2.02.02.07 Case G—Modern Roundabouts
    - Add roundabout control case not previously included
- 5-2.02.03 Effects of Skew
    - Remove direction to pursue 90-degree intersections wherever possible and later encouragement to correct skews greater than 20 degrees in favor of flexible and performance-based guidance
    - Replace old path length adjustment criteria with newer Green Book criteria
    - Add perspective on the increase in pillar size and steepness

Summary of Changes
MnDOT Road Design Manual
19-03

Section 5-4

- Pages 5-4(6) and 5-4(7)
  - Corrected the gore striping direction
  - The figures were cut off on the right side
  - Corrected typo in striped taper

Sheets 2-7(2) and 6-3(3)

- Corrected header information (dates) to correspond with the Transmittal information.

2-7.0 DESIGN PROCEDURES

2-7.01 Design Memorandums

Design Memorandums document project design concepts, standards and exceptions, and indicate whether a project will meet or exceed the minimum standards of the thirteen critical design elements. MnDOT's Highway Project Development Process Handbook contains further clarification on the use of Design Memorandums.

2-7.02 Coordination with Functional Groups

The designer is responsible for properly coordinating with functional groups by contacting them and providing and/or receiving information and guidance concerning specific areas of a project. The functional groups that need to be involved will vary from project to project. Some of the common ones are:

1. Materials;
2. Hydraulics / Water Resources;
3. Bridge;
4. Cooperative Agreements;
5. Utilities Agreements & Permits;
6. Special Provisions;
7. Engineering Cost Data & Estimating;
8. Surveying & Mapping;
9. Traffic Engineering;
10. Maintenance;
11. Construction & Innovative Contracting;
12. Environmental Stewardship;
13. Cultural Resources;
14. Right of Way / Land Management; and
15. State Aid.

2-7.03 Intermodal Coordination

The designer is responsible for properly coordinating with intermodal groups inside and outside the Department by contacting each group that the project may affect in the present or the future. Proper coordination with intermodal groups reduces costly mistakes and may save dollars by combining projects. The intermodal groups are able to determine proper procedures, directions**,** and time frames needed to complete reviews of each project. The intermodal offices to consider contacting for each project are:

1. Freight & Commercial Vehicle Operations;
2. Transit and Active Transportation; and
3. Aeronautics.

2-7.04 Agency/Department Coordination

The designer is responsible for coordinating with the different agencies and/or departments that a project may affect. Some of them are:

1. Federal agencies
    a. United States Department of Transportation
    b. U.S. Coast Guard
    c. U.S. Department of Agriculture, Natural Resources Conservation Services
    d. U.S. Army Corps of Engineers
    e. U.S. Environmental Protection Agency
    f. U.S. Fish and Wildlife Service
2. State agencies
    a. Minnesota Historical Society
    b. Minnesota Department of Natural Resources (DNR)
    c. Minnesota Department of Public Safety (DPS)
    d. Minnesota Pollution Control Agency (MPCA)
    e. Minnesota Environmental Quality Board (MEQB)
    f. Minnesota Board of Water and Soil Resources

3. Counties
4. Municipalities
5. Metro and other regional groups
    a. Metropolitan Council of the Twin Cities
    b. Metropolitan Airports Commission (MAC)
    c. Local planning agencies
    d. Watershed districts
6. National Transportation service groups
    a. American Association of State Highway and Transportation Officials (AASHTO)
    b. Transportation Research Board (TRB)

5-2.0 GENERAL INTERSECTION DESIGN

Even with well-managed access, public road and private driveway intersections are numerous on the system. Therefore, proper design of each intersection and driveway is necessary to optimize safety and operational efficiency within space and funding constraints. Available design treatments include exclusive left-turn and right-turn lanes, improved sight distance, channelized right turns, indirect left turns, higher-level traffic controls, service roads, and ease of ingress/egress, particularly at driveway entrances.

5-2.01 Design Principles

The primary objective of intersection design is to facilitate the movement of transportation modes in a manner that optimizes safety and operational efficiency within the bounds of practicality. This goal can be achieved through the appropriate selection of intersection control and configuration, good geometric design that appropriately sizes design elements, and provision of sufficient sight distance to allow good judgments by users.

Intersections will generally operate more safely and efficiently when designed compactly. The objective is an intersection that allows for the movement of larger vehicles—both common and oversize—but is designed primarily for the safety and comfort of smaller vehicles and non-motorized modes. This often entails the need for multi-point turns and encroachment into adjacent or opposing lanes by commercial traffic, consistent with the Minnesota Commercial Driver's Manual.

Standard intersection sight distance (ISD), discussed below, is based on operational factors. Stopping sight distance (SSD), discussed in Chapters 2 and 3, is considered the minimum sight distance design in the vicinity of intersections and should therefore be provided wherever practical at intersections and driveways as part of new construction.

5-2.02 Intersection Sight Distance

Of all intersection geometrics which may be related to crash frequency, sight distance is most often a contributing factor. Sight distance is fundamental to intersection operation and safety and accordingly deserves special attention. Because intersections are the most prevalent crash location on the road system, robust sight distance provision is sometimes justifiable, especially in addressing crash problem locations or where site specifics complicate the driving task. It must be noted, however, that intersection safety is a multifaceted and not well understood system, of which sight distance is only one component.

As noted above, for new construction, provide at least stopping sight distance (SSD) in the vicinity of intersections and driveways. For improvement projects on existing roads, consider correcting a SSD deficiency near an intersection if the expected improvement in performance warrants the additional construction cost. Take into account the safety history of the site, the expected reduction in number of crashes, the cost of achieving the standard criterion, and the worthiness of the investment compared with other needs and priorities on the system. In the case of poor safety performance at an intersection, study the site to diagnose whether sight distance is a likely contributing factor; lower-cost solutions than sight distance improvement may be available. Where sight distance is to be increased, base the design at least partly on known safety relationships discussed as follows.

Quantitative relationships between available sight distance and safety performance at intersections are published in NCHRP Report 875 (and will be included in future editions of the AASHTO *Highway Safety Manual*). Guidelines and analytical steps for applying this information on projects are provided therein. Use this guidance and data to assess expected safety performance of design options weighed against their respective construction costs. This is particularly useful in situations where improvement of an existing condition is judged necessary as well as where non-standard or robust sight distance provision is contemplated.

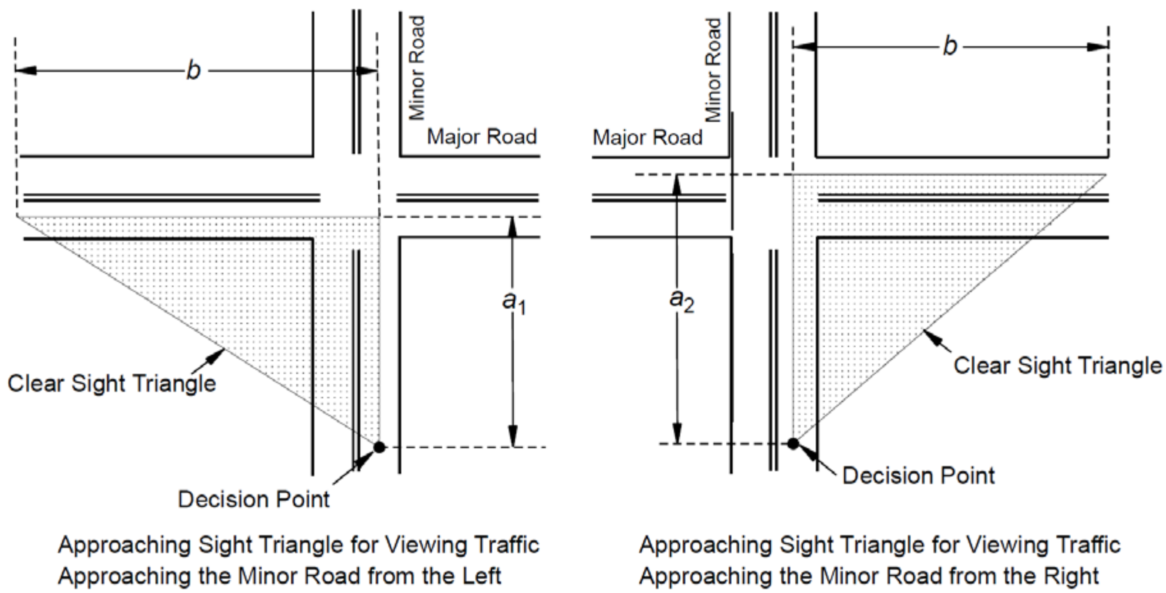
The standard values for intersection sight distance (ISD) that follow are based on allowing potentially conflicting vehicles to perceive each other and act accordingly with minimal operational effect. Its provision is highly encouraged for new construction at intersections with public roads and higher-volume driveway entrances, where practical. The ISD model is founded in the same principles as stopping sight distance but employs modified assumptions based on observed driver behavior at intersections. It is primarily traffic operational in nature and as such should not be applied as representing a minimum degree of safety.

5-2.02.01 Geometric Framework

The basis of intersection sight distance is to provide a line of sight between two vehicles approaching an intersection on their respective legs. This involves establishing a triangular area in each intersection quadrant free of obstructions that might block an approaching driver's view of potentially conflicting vehicles.

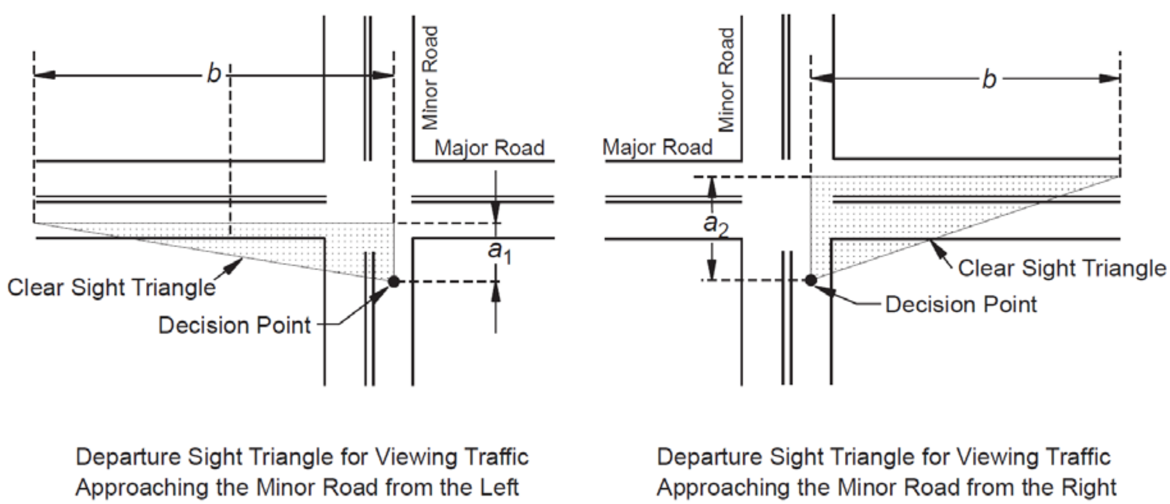
Ideally these so-called sight triangles exist within right of way limits so that terrain and vegetation can be controlled and the placement of obstructing objects precluded. In addition, roadway characteristics such as skew and grade (discussed later in this section), parked cars, guardrail locations, snow storage, signs, walls, fences, and other roadside appurtenances can restrict the line of sight and must be taken into account.

Figure 5-2.02A shows a graphical representation of approach sight triangles. This applies to intersections with no control or yield control on the minor leg (Cases A and C, discussed later in this section) and is based on allowing minor-leg drivers to see potentially conflicting vehicles in sufficient time to avoid a collision by slowing or stopping. The decision point is the location where the driver should begin such a braking maneuver if another vehicle is detected. The triangles' dimensions are as presented in the subsections covering Cases A and C.



APPROACH SIGHT TRIANGLES
Figure 5-2.02A

Figure 5-2.02B depicts departure sight triangles, which provides sight distance sufficient for a stopped minor-road driver to enter or cross the major road. Departure triangles should be provided in each quadrant of each intersection approach controlled by stop or yield signs from which vehicles may enter or cross a major road on which traffic is not required to stop (Cases B and C, later in this section). They should also be provided for some signalized intersection approaches (Case D). The measurements of distances a_1 and a_2 are based on the vertices of the sight triangles (decision points) being located 14.5 feet from the edge of the major road traveled way.



DEPARTURE SIGHT TRIANGLES
Figure 5-2.02B

Approach sight triangles can tend to be more space extensive than departure triangles, particularly on the approach leg. For this reason, selection of intersection control may in some cases be driven by sight triangle availability, including the ability to control vegetation within right of way areas.

Both approach and departure sight triangle conditions may need to be investigated depending on intersection control and site specifics.

Identification of intersection sight distance restrictions must consider both the horizontal and vertical alignment of both roadways as well as any complication resulting from their combination. As discussed in Chapter 3, the assumed height of both the driver's eye and the object to be seen are 3.5 feet above their respective roadway surfaces. The object height is based on a vehicle height of 4 feet, 4 inches (the 85^{th} percentile passenger vehicle height) minus a 10-inch allowance for the portion of the vehicle that needs to be seen in the daytime operating condition in order to be recognized as an object by a driver. In cases where a truck is used as the design approach vehicle, the recommended value for a truck driver's eye height is 7.6 feet.

5-2.02.02 Intersection Control Cases

The recommended dimensions of the ISD sight triangles (a_1, a_2 and b in Figures 5-2.02A & B) depend on the type of traffic control employed at an intersection. Bases, calculation procedures, and tabulated values are presented for each control condition in the following subsections. Additional detail and discussion are available in Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets* as well as *NCHRP Report 383: Intersection Sight Distance*.

Where providing standard ISD in any of these cases is not practical, AASHTO recommends giving consideration to installing advisory speed signing on the applicable major-road approach(es).

5-2.02.02.01 Case A—Intersections with No Traffic Control

Some rural intersections of low-volume roads are uncontrolled. Approach sight triangle geometry is applicable to these situations. Departure sight triangles are not normally considered necessary due to typically very low traffic volumes and corresponding low likelihood that a vehicle stopped due to the presence of conflicting vehicle will encounter another approaching vehicle.

For each dimension a_1, a_2 and b, use the values from Table 5-2.02A based on the design speed of the particular approach leg. These values are based on a perception-reaction time of 2.5 seconds and a braking deceleration rate consistent with the stopping sight distance model. Field observations indicate casual deceleration by approaching vehicles in advance of this sight triangle, even when no potentially conflicting vehicle is detected. For this reason, these dimensions are considered conservative. Where the grade along an intersection approach exceeds 3 percent, the leg of the clear sight triangle along that approach should be adjusted by multiplying the appropriate sight distance by the appropriate adjustment factor from Table 5-2.02B.

Table 5-2.02A
LENGTH OF APPROACH SIGHT TRIANGLE LEG, CASE A

Design Speed (mph)	Length of Leg (ft)
20	90
25	115
30	140
35	165
40	195
45	220
50	245
55	285
60	325
65	365
70	405

Table 5-2.02B
ADJUSTMENT FACTORS FOR ISD BASED ON APPROACH GRADE

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	1.2	1.2	1.2	1.2
-4	1.0	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	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

5-2.02.02.02 Case B—Intersections with Stop Control on the Minor Road

Intersection sight distance criteria for so-called through stop-controlled intersections allows a vehicle stopped at the decision point on the minor road to see a sufficient distance along the major road to turn left or right or to make a crossing maneuver. The standard values are based on these maneuvers not unduly impacting the operation of vehicles on the through roadway. Only departure sight triangles are applicable to this case.

5-2.02.02.02.01 Case B1—Left Turn from the Minor Road

Standard ISD for this case is a departure sight triangle for traffic approaching from the right (Figure 5-2.02B, right-hand illustration). For traffic approaching from the left, the dimension associated with Case B2 provides more than sufficient distance to allow a left turn (refer to 5-2.02.02.02.02).

The length of the departure sight triangle, b, is equal to the distance traveled at the design speed of the major road based on the applicable time from Table 5-2.02C, as adjusted per footnotes. These values provide sufficient time for the minor road vehicle to accelerate from a stop and complete the left-turning maneuver without causing major-road traffic to slow to less than 70 percent of its initial speed. For most purposes, the minor-road vehicle can be assumed to be a passenger car; however, for approaches with unusually high volumes of heavy vehicles, use of the indicated values for trucks may be considered.

Table 5-2.02C
TRAVEL TIMES FOR DETERMINATION OF DEPARTURE SIGHT TRIANGE LEG, CASE B1

Design Vehicle	Time Gap, t_g (s), at Design Speed of Major Road
Passenger car	7.5
Single-unit truck	9.5
Combination truck	11.5

For left turns onto two-way highways with more than two lanes, add 0.5 s for passenger cars or 0.7 s for trucks for each additional lane in excess of one to be crossed by the turning vehicle. Include median width as an equivalent number of 12-foot lanes.

If the approach grade on the minor road is an upgrade exceeding 3 percent, add 0.2 s per percent grade

The length, b, of the departure sight triangle would be determined as:

$$b = 1.47\, V_{major}\, t_g$$

Where:

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

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

Sight distance design for left turns at divided-highway intersections may require consideration of different combinations of design vehicles. For example, if the median is sufficiently wide to store a passenger vehicle but not a combination truck, a calculation of b for the passenger car would not need to adjust the time gap to include the near-roadway width, but that allowance would need to be incorporated if the truck is considered the design vehicle for one or both approaches.

5-2.02.02.02.02 Case B2—Right Turn from the Minor Road

Standard ISD for this case is a departure sight triangle for traffic approaching from the left (Figure 5-2.02B, left-hand illustration), calculated in the same manner as for Case B1 but with slightly different time gaps. Such provision should not be considered necessary where right turns are accommodated with channelized free-right geometry.

Field observations indicate that drivers making right turns generally accept slightly shorter gaps than those accepted in making left turns. For this reason—and to account for not having to cross one lane as part of the maneuver—the travel times in Table 5-2.02D are to be used for right turns, as adjusted per footnote. They are reduced by 1.0 s from those applied to the left-turning condition.

Table 5-2.02D
TRAVEL TIMES FOR DETERMINATION OF DEPARTURE SIGHT TRIANGE LEG, CASES B2 & B3

Design Vehicle	Time Gap, t_g (s), at Design Speed of Major Road
Passenger car	6.5
Single-unit truck	8.5
Combination truck	10.5

If the approach grade on the minor road is an upgrade exceeding 3 percent:
Add 0.1 s per percent grade for right turns
Add 0.2 s per percent grade for crossing maneuvers

For a crossing maneuver of a road with more than two lanes, add 0.5 s for passenger cars and 0.7 s for trucks for each additional lane to be crossed. Include median width as an equivalent number of 12-foot lanes.

5-2.02.02.02.03 Case B3—Crossing Maneuver from the Minor Road

In most situations, it can be assumed that the departure sight triangles from Cases B1 and B2 are more than adequate for minor-road vehicles to cross the major road. In the case of steep approach grades or wide major roadways, however, the time gaps in Table 5-2.02D, adjusted per the footnotes pertaining to the crossing maneuver, can control the design and should be calculated.

5-2.02.02.03 Case C—Intersections with Yield Control on the Minor Road

Since a yield condition allows drivers to enter an intersection without stopping if there are no conflicting vehicles present, an intersection with yield control on the minor-road approach must be designed for this eventuality as well as for minor-road traffic to stop. For this reason, both approach sight triangles and departure sight triangles are applicable to these intersections.

Similar to intersections with no control, standard ISD for yield control entails establishing approach sight triangles in both directions—as depicted in Figure 5-2.02A—for each yield-controlled approach. Unlike uncontrolled intersections, however, two separate pairs of approach sight triangles are necessary—one to accommodate the crossing maneuver and one for left and right turns onto the major road. This is due to unique driver behavior observed at yield-controlled intersections. Both sets of sight triangles should be checked for potential sight obstructions.

As discussed in 5-2.02.01, yield-controlled intersections generally entail greater intersection sight distances than intersections with stop control due to the inclusion of approach sight triangles. If ISD associated with yield control is not available, consideration may be given to stop control on that basis.

5-2.02.02.03.01 Case C1—Crossing Maneuvers

The length of the near-side approach sight triangle, a_1, is provided in Table 5-2.02E along with its associated travel time t_a. Adjust the table value per the footnote for approaches on steep grades. These distances have the same basis as for Case A except that, based on field observations, minor-road vehicles that do not stop are assumed to reduce their approach speed somewhat less than with an uncontrolled approach.

Table 5-2.02E
LENGTH OF APPROACH SIGHT TRIANGLE LEG AND ASSOCIATED TRAVEL TIME, CASE C1

Design Speed (mph)	Length of Leg, a_1 or a_2 (ft)	Travel Time from Decision Point to Major Road, t_a (s)
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

For minor-road approach grades exceeding 3 percent, multiply table values by the appropriate adjustment factor in Table 5-2.02B.

The length of the major-road approach sight triangle, b, provides sufficient time for the minor-road vehicle to travel from the decision point to the intersection based on the travel-time assumptions above and to cross and clear the intersection at its assumed reduced speed. This length of time and associated major-road dimension are computed using the following equations:

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

$$b = 1.47\, V_{major}\, t_g$$

Where:

t_g = travel time to reach and clear the major road (s), no less than the time for crossing a roadway from a stop given in Table 5-2.02D

b = length of approach sight triangle leg along the major road (feet)

t_a = travel time to reach the major road from the decision point for a vehicle that does not stop (s)—from Table 5-2.02E, adjusted per footnote for approach grade

b = width of intersection to be crossed (feet)

L_a = length of design vehicle (feet)

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

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

As noted above, the value of t_g must equal or exceed the travel time to cross a roadway from a stop, as shown in Table 5-2.02D, adjusted per footnotes. This value commonly controls the design when the minor road design speed is greater than 50 mph.

5-2.02.02.03.01 Case C2—Left- and Right-Turn Maneuvers

The length of the approach sight triangle leg along the minor road (distance a_1 in Figure 5-2.02A) to accommodate left and right turns without stopping is approximately 75 feet. This is based on drivers slowing to a turning speed of 10 mph.

The approach sight triangle lengths along the major road, b, are similar to the values for stop-controlled intersections in Cases B1 and B2—from Tables 5-2.02C & D respectively—but increased by 0.5 s over those values. This reflects 3.5 s of additional time needed for approach vehicles to travel from the decision point to the intersection minus 3.0 s due to not having to accelerate from a stop under the yield condition.

Departure sight triangles are appropriate in order to account for minor-road vehicles that stop at the yield sign to avoid conflicts with major-road vehicles. These need not be calculated, however, since approach sight triangles for turning maneuvers under yield control are inherently larger.

5-2.02.02.04 Case D—Intersections with Traffic Signal Control

A basic requirement for all signal-controlled intersections is that drivers must be able to see the control device soon enough to perform the action it indicates. Lines of sight should also be provided such that the first vehicle stopped on each approach is visible to the driver of the first vehicle stopped on each of the other approaches. Additionally, except for where right turns on a red indication are disallowed, departure sight triangles for Case B2 should be checked. If the signal is to be placed on two-way flashing operation under off-peak or nighttime conditions, other appropriate Case B sight triangles would be in order.

5-2.02.02.05 Case E—Intersections with All-Way Stop Control

The first stopped vehicle on each approach should be visible to the drivers of the first stopped vehicles on each of the other approaches.

5-2.02.02.06 Case F—Left Turns from a Major Road

Intersection sight distance to accommodate left turns by a vehicle on the major road is the distance traversed at the design speed of opposing traffic in the travel times given in Table 5-2.02F. They are based on the assumption that the turning vehicle has come to a stop. Such distances allow the maneuver to be completed without unduly affecting the operation of the oncoming vehicle. These exceed the corresponding stopping sight distances which, as discussed previously, should be applied as a minimum design criterion for new construction in the vicinity of intersections. As footnoted, adjust the time gaps for additional major-road lanes and/or median to be crossed by the turning vehicle.

Table 5-2.02F
TRAVEL TIMES FOR USE IN CALCULATING ISD, CASE F

Design Vehicle	Travel Time (s) at Design Speed of Major Road
Passenger car	5.5
Single-unit truck	6.5
Combination truck	7.5

For a crossing maneuver of a road with more than two lanes, add 0.5 s for passenger cars and 0.7 s for trucks for each additional lane to be crossed. Include median width as an equivalent number of 12-foot lanes.

If sight distance for Case B or Case C has been provided for each minor-road approach, the Case F criteria will generally be satisfied. This may not be true of 3-leg intersections, atypical configurations, or driveways where departure sight triangles are not provided, in these situations, the Case F condition should be checked.

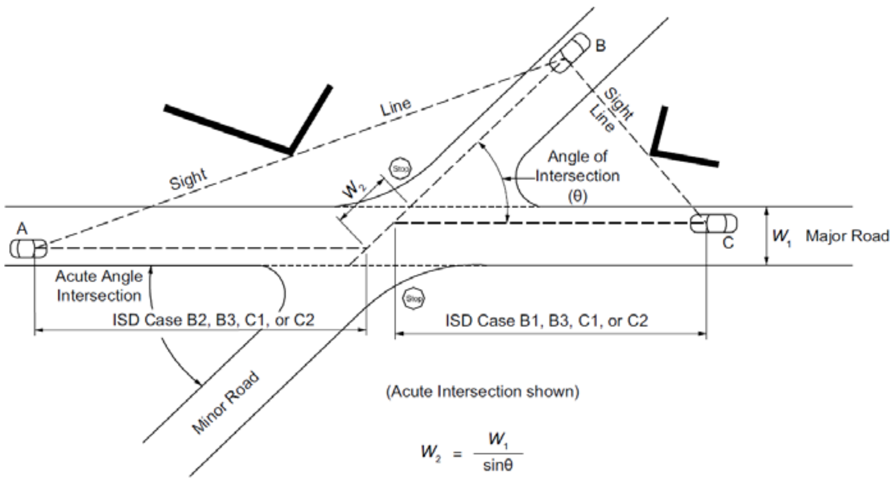
5-2.02.02.07 Case G—Modern Roundabouts

Drivers entering a roundabout need to see potentially conflicting vehicles along the upstream portion of the circulatory roadway as well as those entering from the immediate upstream approach. Refer to *NCHRP Report 672*, which presents a procedure for determining the associated sight lines and distances.

Unique to roundabouts is the concept of limiting sight lines to no more than needed for the driver to decide when to enter the intersection—the driver’s sole operational task. Allowing a view of the entire circulatory area can potentially distract from that task as well as encourage higher circulating speeds, both of which can affect the safety and efficiency of the intersection. The standard practice of elevating the central island restricts sight lines in this fashion.

5-2.02.03 Effects of Skew

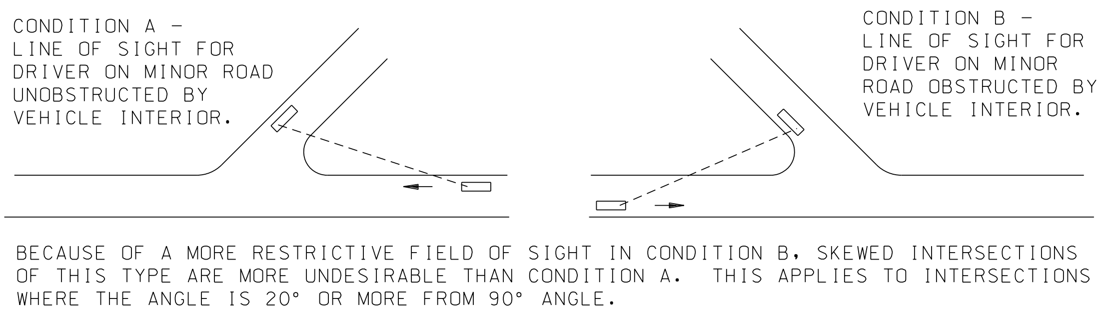
Oblique-angle intersections entail increased travel path lengths for some turning and crossing maneuvers. Intersection angles deviating from a right angle by more than 15 degrees should be analyzed and sight triangle distances increased if an actual path length increases by one or more equivalent lanes on account of skew. This adjustment is made by dividing the lateral width of the lanes and/or median being crossed by the sine of the intersection angle, as shown in Figure 5-2.02C. Refer to discussion, tables and footnotes for each intersection control case for baseline travel times and assumptions, mindful that the time gaps for each case already account for crossing of one lane.



SIGHT TRIANGLES AT SKEWED INTERSECTIONS
Figure 5-2.02C

In the obtuse-angle quadrant of a skewed intersection, the angle between the approach leg and the sight line is small, requiring only a small head movement to see the entire sight triangle. In contrast, the acute-angle quadrant demands considerable head movement to see the sight triangle. For this reason, it is recommended that Case A not be applied to acute quadrants where the intersection angle is less than 60 degrees; rather, compute the sight triangle dimensions using Case B criteria in these situations.

Pillars supporting vehicles' roofs have increased in size and steepness over the years for reasons of crashworthiness and aerodynamics, but the larger pillars entail more prominent sightline restrictions and blind spots from the driver's perspective. The sightline restriction varies depending on which side the minor road intersects, as illustrated in Figure 5-2.02D. This factor should be considered when designing an intersection. For existing sites, an individual analysis--including review of the crash history--is necessary to evaluate the potential for realignment and which alternative design may be most cost-effective.



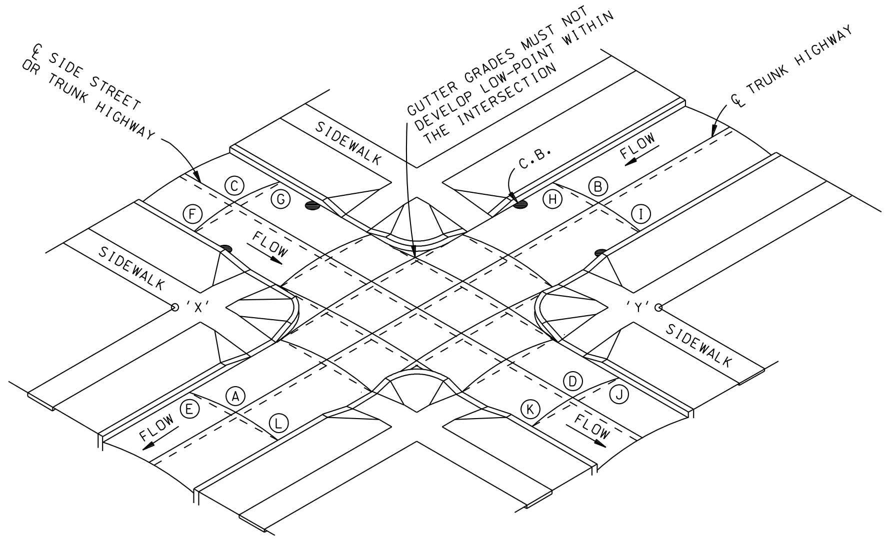
SKEWED INTERSECTION - LINE OF SIGHT
Figure 5-2.02D

5-2.03 Effect of Grades

The grades of intersecting highways should be as level as possible. Grades approaching an intersection affect the normal stopping and accelerating distances. Adjustments in these distances are needed if the grade exceeds +/- 3%. See chapter 9 of AASHTO's "A Policy on Geometric Design of Highways and Streets" for details. The gradient within the intersection should be +/- 0.5% for the highway section needed for storage. Between 1.5 and 2.0% are acceptable to match cross slope of pavement.

Attention must be given to adjusting the cross sections of the two intersecting highways to achieve a smooth crossing and proper drainage. Figures 5-2.03A, B, and C demonstrate three methods of accomplishing this for various combinations of closed and open drainage and the functional class of the intersecting highways.

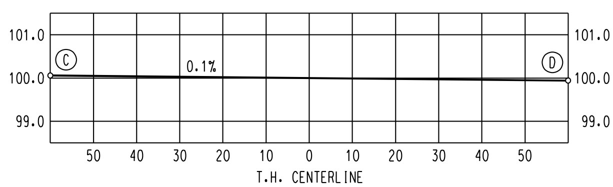
Special attention should be given to signalized intersections where high speeds are possible on both crossing roadways. Figure 5-2.03A, in combination with an appropriate superelevation rate of change (1:100 maximum slope), should be used as a guide when designing a signalized intersection. Design of more complicated intersections may require contouring the intersection at appropriate contour intervals to develop proper drainage and a smooth roadway crossing.



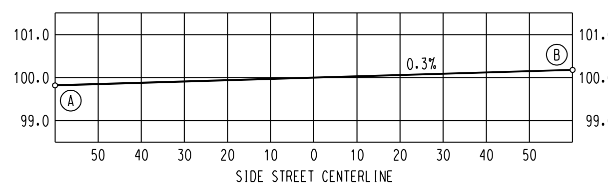
NOTES: IN ORDER TO AVOID EXCESSIVE SIDEWALK AND BOULEVARD GRADIENTS, POINTS X AND Y MUST BE CONSIDERED WHEN LAYING GUTTER GRADES. IF EXCESSIVE WALK GRADIENTS ARE ENCOUNTERED, ADJUST GUTTER GRADIENTS (E-F AND I-J) TO ALLEVIATE THE CONDITION IN PREFERENCE TO VARYING THE HEIGHT OF CURB.

AREAS BETWEEN DASHED AND SOLID LINE DO NOT REPRESENT THICKNESS OF PAVEMENT BUT INDICATE RELATIVE HEIGHTS OF CROWN.

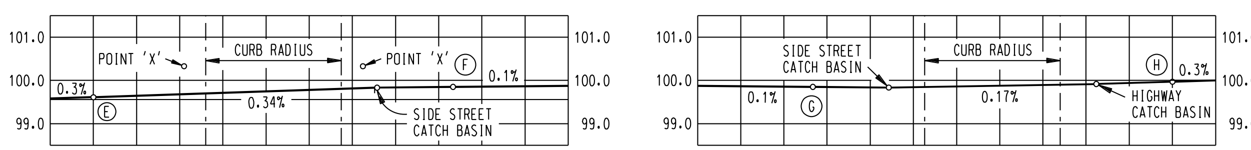
SIDE STREET CENTERLINE PROFILE

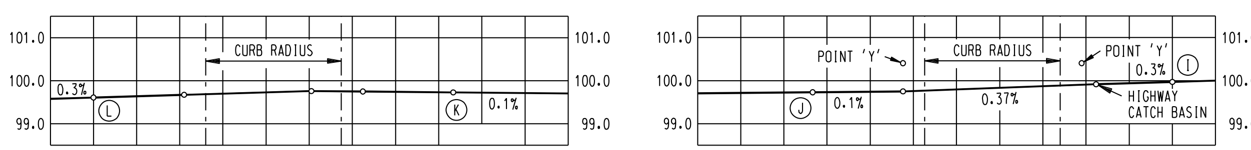


TRUNK HIGHWAY CENTERLINE PROFILE



Gutter Profiles





* THE INTERSECTION OF SIDE STREET AND TRUNK HIGHWAY CENTERLINE PROFILES IS DESIGNATED AS ELEVATION 30.0 FOR ILLUSTRATIVE PURPOSES.

METHOD A: WARPING TRUNK HIGHWAY AND SIDE STREET CROWNS. UNDERGROUND DRAINAGE REQUIRED.

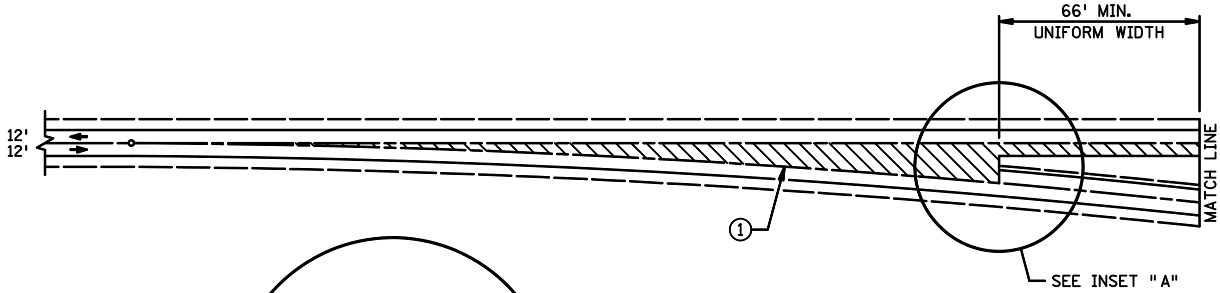
URBAN INTERSECTION - METHOD A
Figure 5-2.03A

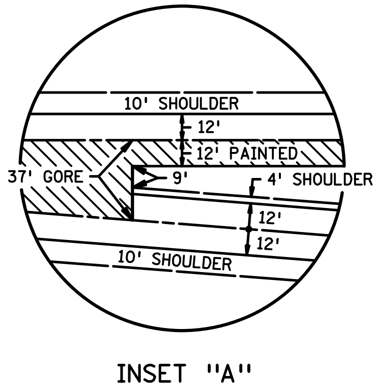
5-4.01.07 Transition from Divided to Single Highway

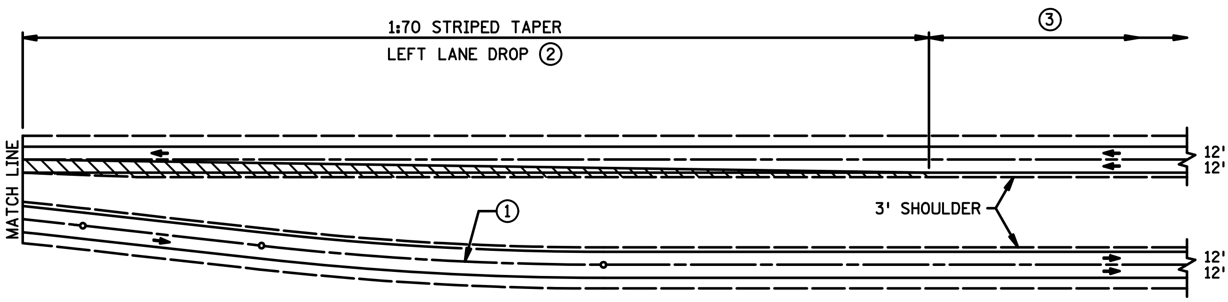
A divided highway transition to a single highway provides for terminating a four-lane divided highway, or in reverse, starting one. It is important that this transition be made easy for the driver to understand and drive. Generally, the executed maneuvers done by the drivers in the transition section are done at normal highway speeds.

The two designs for the transition section shown in Figure 5-4.01D and Figure 5-4.01E are different in the sense that one is a left lane drop and the other is a right lane drop. Each design can be used as the situation may require in any one existing condition. However, in new construction or reconstruction projects, the right lane drop is preferred because, in case of emergency, cars in that lane can use the right shoulder as a refuge.

The location of the transition section should be some distance away from any major roadway. If a transition section must be constructed near a crossroad, then as a minimum, a distance as required by the Traffic Manual for proper signing should be provided. There should not be any entrances to adjoining properties in the tapered lane drop area, and in particular, not in an area 250 ft upstream and downstream from the nose. It is also important to keep in mind that any crossroad in the two-lane section should be far enough from the gore to have the right-turn lane developed in the two-lane section beyond the gore.

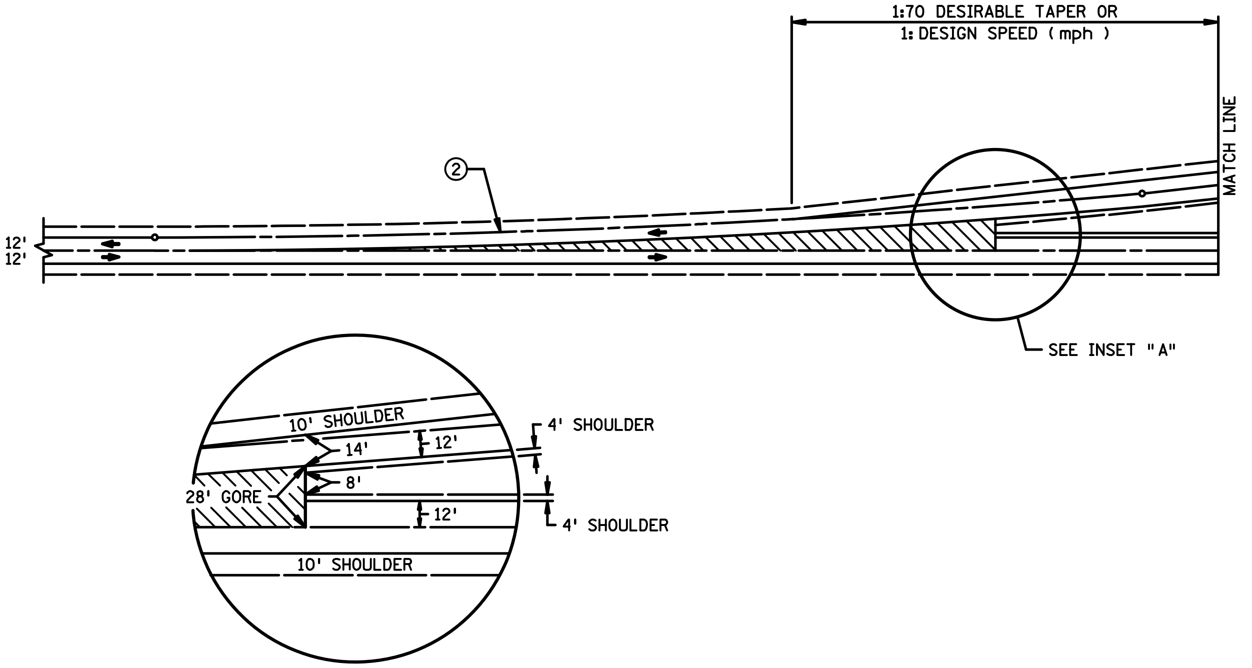




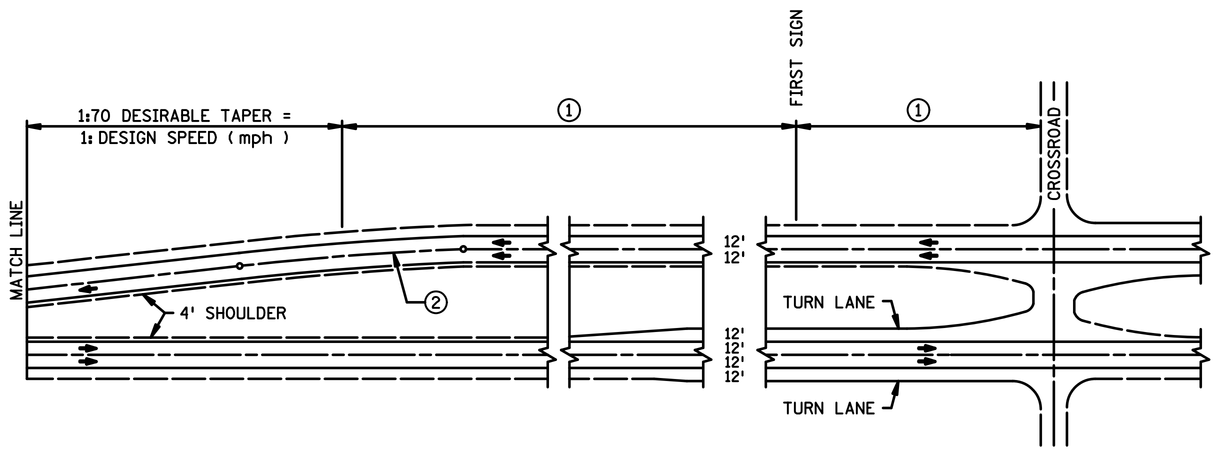


NOTES:
1 0° 30' TO 2° CURVE.
2 RIGHT LANE DROP IS PREFERABLE.
3 SIGNING INTERVALS PER DISTRICT TRAFFIC ENGINEER.

RURAL DIVIDED HIGHWAY TO SINGLE HIGHWAY TRANSITION
Tangent Approach to Single Highway, Left-Lane Drop
Figure 5-4.01D



INSET "A"



NOTES:
1 SIGNING INTERVALS PER DISTRICT TRAFFIC ENGINEER.
2 0° 30' TO 2° CURVE.

RURAL DIVIDED HIGHWAY TO SINGLE HIGHWAY TRANSITION
Curve Approach to Single Highway, Right-Lane Drop (Preferred)
Figure 5-4.01E

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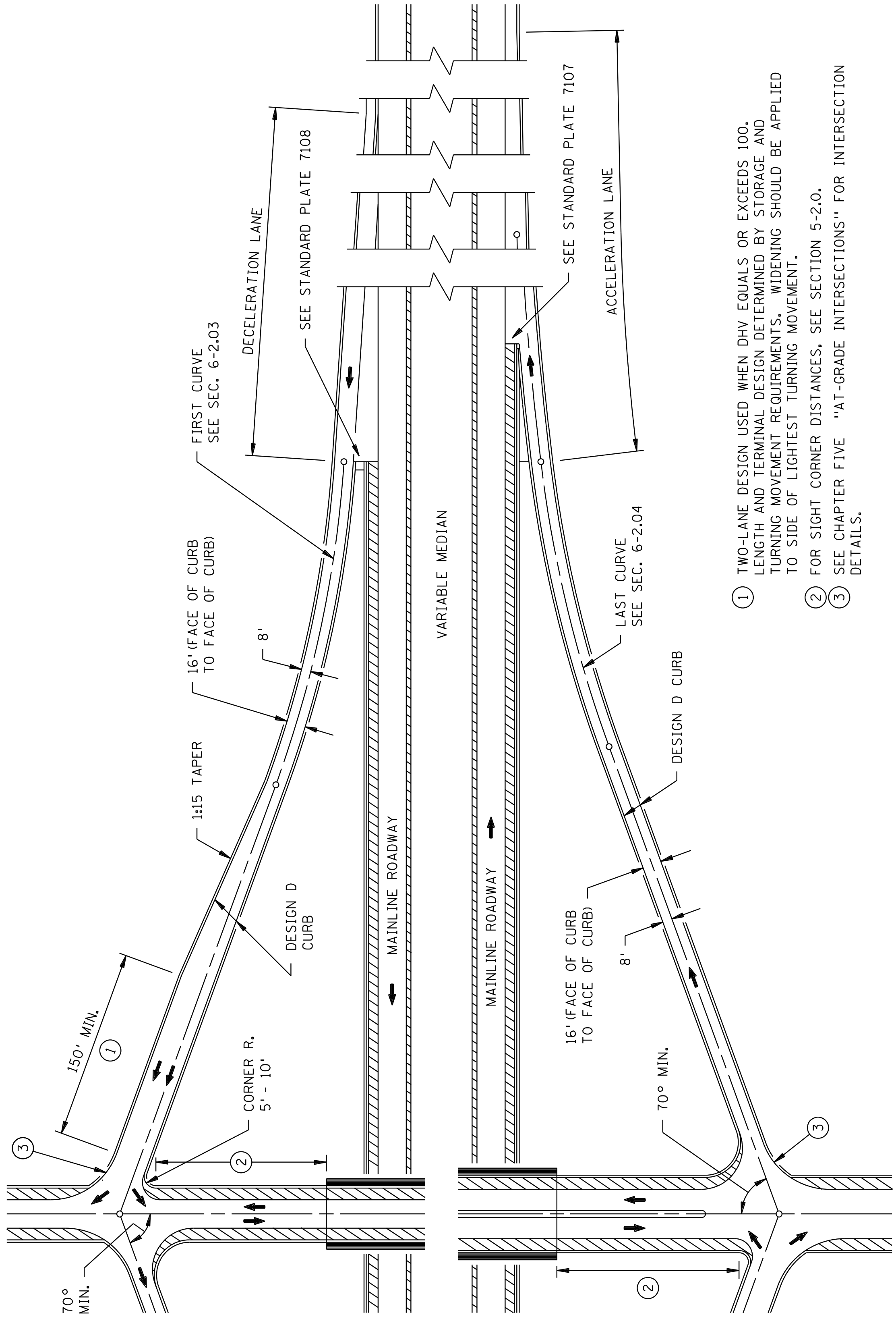

DIAGONAL RAMP DESIGN
DIAMOND INTERCHANGE - URBAN
Figure 6-3.02B

6-3.04 Design Elements

6-3.04.01 Design Speed

The design speed of the ramp proper should conform to the expectations of drivers and fit the constraints and topography of each location. In practice this involves designing the curve adjoining the mainline terminal to a certain percentage of the mainline design speed, depending on context, degree of constraint, and construction cost. The other portions of the ramp are designed based loosely on an assumed speed profile along its length. For ramps that terminate at an intersection, uniform deceleration to a stop condition is usually appropriate. Direct and outer connections are most often designed for a constant speed. For semi-direct connections, the portion between the mainline terminals is usually designed to a somewhat lower speed than the terminal curves, typically dictated by site specifics and interchange configuration.

For a given mainline design speed, Table 6-3.04A gives the corresponding ranges of ramp design speed and associated minimum radius, applicable to the first/last curve adjoining the mainline terminal (but not the transitional curves to/from the main curve).

1. On diagonal ramps (such as ramps in diamond or parclo interchanges), the minimum design speed is the value from the lower range of Table 6-3.04A. In all but the most constrained situations (buttonhook configurations and urban core locations, for example), the desirable minimum design speed is the value from the middle range of the same table. To avoid excessive interchange footprints, design speeds in the high range are not recommended for diagonal ramps having reversing curvature, particularly those in parclos.
2. For loops, AASHTO recommends a design speed no less than 20 mph (110-ft radius) for use with high-speed highways and encourages above-minimum designs in less constrained locations. Radii between 140 ft (22.5 mph) and 170 ft (25 mph) have exhibited good performance with typical freeway design speeds (50 mph to 70 mph) where spiral or robust circular transition treatments are used (see 6-3.04.02). A maximum practical radius is 250 ft (30 mph), above which space requirements and travel times become excessive.
3. On semi-direct connections (as shown in Figure 6-1.03K) as well as outer connections in cloverleaf and semi-directional interchanges, the minimum design speed is the value from the middle range of Table 6-3.04A. This also applies to two-lane semi-direct connections.
4. A direct connection (as shown in Figure 6-1.03L) often carries a mainline route or has comparable significance or traffic demand. In these cases, a uniform design speed along its entire length based on guidelines for mainline highways may be appropriate. A value somewhat lower than for an open-road condition is often justified, however, to fit configuration and constraint. The minimum design speed for any direct connection is the value from the middle range of Table 6-3.04A, not less than 40 mph.

Refer to Chapter 3 for criteria pertaining to superelevation rates and transitions. Generally apply superelevation to the first/last curve per Table 3-3.02A and the selected ramp design speed; however, curves less than 200 feet in length may be sloped at the normal cross slope rate to avoid near-continuous transitioning though the curve. To simplify design, secondary curves on diagonal ramps should be superelevated only as necessary to limit side friction to f_{max}, based on Figure 3-3.03A and an assumed speed at that point on the ramp. Loops should always receive full superelevation (0.06 to 0.08 (ft/ft, m/m)).

Table 6-3.04A
RAMP DESIGN SPEEDS

Highway Design Speed (mph)	40	45	50	55	60	65	70	75
Ramp Design Speed (mph)								
High Range (85%)	35	40	45	50	50	55	60	65
Middle Range (70%)	30	30	35	40	45	45	50	55
Low Range (50%)	20	22.5	25	27.5	30	30	35	40
Corresponding Minimum Radius (ft) Based on 0.08 (ft/ft) Maximum Superelevation								
High Range	350	465	600	760	760	960	1,200	1,500
Middle Range	250	250	350	465	600	600	760	960
Low Range	110	140	170	210	250	250	350	465