



FDM 11-10-1 Basic Criteria

November 15, 2022

1.1 Design Year

The design years for projects are normally 20 years from the date projects are proposed to be opened to traffic. Shorter design periods may be used when highways are to be constructed in stages or designed for shorter pavement improvement lifespans.

1.2 Traffic

Cooperate with the region's planning staff to develop design traffic data. Traffic data includes current and design year average daily traffic, design hourly volumes, directional distributions, and the percentages of heavy vehicles expected in the design year. Normally trucks and buses are the heavy vehicles considered as influencing highway capacities. Also, consider heavy recreational vehicles on certain routes. Include bicycle and pedestrian counts when requesting intersection traffic counts.

1.3 Highway Capacity

Capacity is an important factor in highway design and operation. Through capacity analyses, proposed highways can be designed to operate at predicted traffic volumes without exceeding pre-selected levels of service. Early programming and scoping processes will be the first evaluations to determine if capacity issues will be addressed or not and to what levels of scope. Generally,

- Perpetuation projects will generally not explore capacity improvements,
- Rehabilitation projects will explore limited capacity improvements in the form of incremental improvements or safety mitigation measures as defined in the projects purpose and need,
- Reconstruction-Type Rehabilitation projects will explore more project segments/locations for incremental improvements, as defined in the projects purpose and need, and
- New Construction-Type Modernization projects will fully explore capacity expansion improvements as defined in the projects purpose and need.

Refer to [FDM 11-5-3](#) for highway capacity procedures.

1.4 Functional Classification

Functional classification is the process by which streets and highways are grouped into classes or systems according to the character of service they are intended to provide. The basic functional systems used in highway planning are arterials, collectors, and locals. Using national classification terminology, these systems are sub-classified based on the trips served, the areas served, and the operational characteristics of the streets or highways. These systems are detailed on Wisconsin's current Functional Classification Systems Maps:

<https://wisconsindot.gov/Pages/projects/data-plan/plan-res/function.aspx>

1.5 Design Speeds

According to the 2018 GDHS¹, "Design speed is a selected speed used to determine the various geometric design features of the roadway. The selected design speed should be a logical one with respect to the anticipated operating speed, topography, the adjacent land use, modal mix, and the functional classification of the roadway." Selections of design speeds are very important because these choices set limits for curvatures, sight distances, clear zones, and other geometric and cross-sectional features.

Speed measurements on highways of different design speeds and various traffic volumes typically show wide ranges of actual vehicle operating speeds. The 85th percentile speed is used as the primary basis of establishing posted speed limits, and by extension, design speeds. The 85th percentile speed is the speed at or below which 85 percent of free-flowing vehicles are traveling at. See [TEOps 13-5-12](#) for more information. When new or modified speed limits are proposed on state or locally maintained roadways, coordination with the Region traffic unit is required. Any modifications from established speed limits or statutory speed limits shall be

¹ A Policy on Geometric Design of Highways and Streets, 7th edition, AASHTO, 2018, [p. 2-23](#)

documented with a speed zone declaration as per [TEOpS 13-5-10](#).

It is commonplace to select a design speed above the posted regulatory speed as operating speeds may exceed posted speed limits. Table 1.1 provides corresponding design speeds with typical posted speeds. Careful consideration should be given when selecting a design speed as geometric elements within a facility such as wide medians or shoulders may influence a driver's behavior to exceed posted limits.

Table 1.1 Design Speeds vs. Typical Posted Speeds

| Posted Speed (mph) | Design Speed (mph) |
|--------------------|--------------------|
| 25 | 25-30 |
| 30 | 30-35 |
| 35 | 35-40 |
| 40 | 40-45 |
| 45 | 45-50 |
| 50 | 50-55 |
| 55 | 55-60 |
| 60 | 60-65 |
| 65 | 65-70 |
| 70 | 70-75 |

On redesigned or newly designed roadways, design alignments with purposeful, curvilinear features to promote operating speeds that are compatible with the chosen design speeds. Also, give special consideration to corridor consistency and functional class, when selecting appropriate design speeds. See the design criteria tables in [FDM 11-15-1](#) and [FDM 11-20-1](#).

Existing two-lane roadways being expanded to four lanes are good candidates for 70-75 MPH design speeds where the following conditions exist:

1. The planned highways meet the requirements for freeways or expressways as defined in [Section 346.57\(1\)](#) of the Wisconsin Statutes.
2. The cross sections of the roadways will be divided (i.e., opposing traffic will be separated by medians or traffic barriers).
3. If, when checked with the regional traffic unit, it is confirmed that the highway segments are long enough to allow the practical use of the higher design speeds.
4. The median widths equal or exceed the clear zone widths required for 70 mph design speeds.

Lower design speeds may be considered as part of the project. This shall be based on the existing and anticipated roadway use and include factors such as operating speeds, safety considerations, land use, access management, bicycle and pedestrian activity, and enforceability. See [TEOpS 13-5-1](#) for more information. Selections of design speeds shall be mutually agreed on between the planning, traffic and project development sections.

Regardless of the design speeds provided in [FDM 11-15-1](#) and [FDM 11-20-1](#), the basis for selections of design speeds should be fully documented in the Final Scope Certification (FSC) if required based on the improvement types. If the selected design speeds equal or exceed the legal speed limits, statements to that effect will suffice. Otherwise, this documentation shall include discussions of the road characteristics that relate to operating speeds plus characteristics of the abutting segments of roads, and statements about any advisory or regulatory speed signing in place, and discussions about practical operating speeds on the affected and abutting segments of roads; or other logic that explains the basis

for the selections.

Design Justifications (DJs) to the design speed policy are not required on spot improvement projects. In these contexts, spot improvements are defined as projects less than 0.5-mile long (e.g., a bridge replacements). Instead, choose design speeds that are consistent with both existing conditions and the planned developments of the adjacent sections of highways. If these result in project design speeds that are less than the posted or statutory speeds, then mitigate these to the extent possible by appropriate advisory signing, or other means.

Project Type (Perpetuation, Rehabilitation and Modernization) should not affect selection of design speeds. Most projects should incorporate project wide design speeds compatible with the posted speed limits but may retain existing features having lower than design speed ratings if justified through the FSC or by acceptance of DJs as needed. For further information about design speeds, see the section titled "Speed" in the 2018 GDHS².

FDM 11-10-5 Geometric Elements

November 15, 2022

5.1 Sight Distances

A primary feature of highway design is the arrangement of the geometric elements so that there is adequate sight distance for safe and comfortable vehicle operation. Sight distances are considered in terms of stopping sight distances, decision sight distances, passing sight distances, and intersection sight distances.

For the purposes of driveway permitting, driveway sight distances should equal intersection sight distances (it is also recommended that driveway sight distances be evaluated on Modernization projects).

Consistent quality designs require that sight distances be evaluated for the entire projects as wholes, rather than looking at isolated lengths of roadways. Adjustments in alignments and profiles should be evaluated to produce improvements in the availability, distribution, and balance of sight distances along the routes.

Use upper minimum design sight distance criteria categories and values on Modernization projects in most cases. Lower minimum design sight distances are available to use in appropriate situations in which they can be justified, documented, and approved through the SCDs or in DJs. Below minimum sight distance criteria categories and values can be retained on Perpetuation, Rehabilitation, and some Reconstruction-Type Modernization projects if the SCD results show no safety issues related to them. Otherwise, use of sight distance criteria categories and values below minimum need to be justified, documented, and approved through DJs. See [FDM 11-3-5](#) for guidance on design criteria.

5.1.1 Stopping Sight Distances (SSDs); Decision Sight Distances (DSDs)

5.1.1.1 Stopping Sight Distances (SSDs)

Stopping Sight Distance (SSD)³ is the length of roadway ahead that is visible to drivers that is sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. It is the sum of two distances:

1. Brake reaction distance - the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied
2. Braking distance - the distance needed to stop the vehicle from the instant brake applications begin. Stopping distance is calculated using the 90th percentile reaction time of 2.5 seconds and the 90th percentile deceleration rate of 11.2 feet/s² on wet pavement.

[Attachment 5.1](#) and [Table 3-1, page 3-4, 2018 GDHS](#) show the required stopping sight distances for design speeds from 25-70 mph. Consider adjustments to SSDs for downgrades and using values exceeding those shown in [Attachment 5.1](#) (See [Table 3-2, page 3-6, 2018 GDHS](#)). (GDHS hyperlink is only available to WisDOT staff.)

Stopping sight distances are used when vehicles are traveling at design speeds on wet pavements when individual clearly discernable objects or obstacles are presented in the roadways. Use the same SSDs for trucks and cars because recent data shows that the braking distances of trucks and passenger cars on wet pavements

² A Policy on Geometric Design of Highways and Streets, 7th edition. AASHTO, 2018, [Section 2.3.6](#) Speed, pages 2-21 thru 2-27.

³ A Policy on Geometric Design of Highways and Streets, 7th edition. AASHTO, 2018, Pages 3-1 thru 3-19, Chapter 3 Elements of Design, [Section 3.2 Sight Distance](#)

are nearly equal⁴.

5.1.1.2 Decision Sight Distances (DSDs)

Decision Sight Distance (DSD) is the distance needed for drivers to detect an unexpected or otherwise difficult-to-perceive information source or condition in a roadway environment that may be visually cluttered, recognize the condition or its potential threat, select an appropriate speed and path, and initiate and complete the maneuver safely and efficiently. There are 5 categories of avoidance maneuvers identified in 2018 GDHS:

- A: Stop on rural road
- B: Stop on urban road
- C: Speed/path/direction change on rural road
- D: Speed/path/direction change on suburban road
- E: Speed/path/direction change on urban road

Design values for DSDs are shown in [Attachment 5.1](#) and [Table 3-3, page 3-8, 2018 GDHS](#).

Decision sight distances are to be considered when conditions are complex, driver expectancies are different for the situations, or visibility to traffic controls or design features are impaired. Complex situations create unsafe or inefficient operations because there is more information for drivers to process. Because of this, drivers need increased perception reaction times to make the proper decisions. These increased times can be especially beneficial for older drivers, because they are involved in disproportionate numbers of crashes where there are higher than average demands imposed on driving skills.

5.1.1.3 Applications of Stopping Sight Distances (SSDs) and Decision Sight Distances (DSDs)

In computing and measuring sight distances along roadways, the heights of the driver's eyes are 3.5 feet above the pavement surfaces, and the height of the object to be seen by the driver is 24-inches. A 24-inch object is equivalent to the taillight height of a passenger car because stopping is generally in response to another vehicle or large hazard in the roadway⁵.

There are 2 criteria categories for sight distances along roadways:

- *Category 1* – must meet the Lower Minimum for SSD to a 24-inch object;
- *Category 2* – provide for DSD for avoidance maneuvers C to a 24-inch object, but must meet the Lower Minimum for SSD to a 24-inch object;

Application and Sight Distance Boundaries, as well as some examples, are found in [Attachment 5.2](#). Application of design criteria are based on the complexity of the driving conditions that can be expected at particular locations. Category 1 applies to the least complex locations and is the default requirement for locations where the other categories don't apply. Category 2 applies to the most complex locations.

Designers may use Decision Sight Distances at other locations that are not listed in [Attachment 5.2](#) if they judge them to be necessary. These need to be evaluated in the SCDs. Some examples of locations where Decision Sight Distances may be appropriate are where:

- Complex operations or design features exist, including abrupt or unusual alignment changes;
- Detour Approaches;
- High-speed high-volume urban arterials with considerable roadside friction.

For Perpetuation, Rehabilitation and Reconstruction-Type projects, the SCDs will determine the safety performance of existing sight distances and if no safety problems exist will not require any further evaluations of sight distances. Otherwise, evaluate sight distances on both horizontal alignments and vertical alignments for physical obstructions along the sight distance lines-of-sight (e.g., roadside structures, crest vertical curves, overpasses). Sag vertical curves are not considered physical obstructions (see [FDM 11-10-5.4.2](#) for a discussion of sag vertical curves). Evaluate sight distance design criteria in both directions of travel on roadways because the sight distance categories might not be the same for both directions of travel.

When sight distances require evaluation, these criteria pertain to all roads. It is encouraged to provide sight distances that equal or exceed Upper Minimum values, particularly on two-lane bi-directional roads where

⁴ *Commercial Truck and Bus Safety Synthesis Program Synthesis 3: Highway / Heavy Vehicle Interaction*. Transportation Research Board, 2003. http://onlinepubs.trb.org/Onlinepubs/ctbssp/ctbssp_syn_3.pdf, p.22

⁵ *NCHRP Report 400: Determination of Stopping Sight Distance*. TRB, National Research Council, 1997., pp. 44-45; 74-76

passing sight distances are provided if they are economically obtainable. Providing less than minimum values requires approved SCDs or DJs per [FDM 11-38](#), [FDM 11-1-20](#) or [FDM 11-4-10](#) depending on the improvement types and situations.

5.1.1.4 Sight Distances on Stop Sign Controlled Approaches

The minimum sight distance design criteria along roadways approaching stop signs is stopping sight distance (SSD) based on the design speeds of the roadways to a 24-inch object, depending on the sight distance category (see [FDM 11-10-5.1.1.3](#) and [Attachments 5.1](#) and [5.2](#)). The horizontal and vertical sight distance criteria are as described for the sight distance category.

Another consideration, in addition to stopping sight distances, is stop sign visibility. Road users need to perceive stop signs for sufficient distances to respond to them. Drivers approaching stop signs typically decelerate over greater distances than SSDs ⁶. This allows more gradual decelerations than are used for SSDs. To achieve these, make sure that the stop signs are perceptible from the upstream functional lengths of intersections (see [FDM 11-25-1](#)). Although it is preferable that the stop signs be visible, it may be necessary to provide other traffic control devices, such as “Stop Ahead” signs, if there are none.

Also, see [FDM 11-10-5.2.2](#) “Horizontal Curves on Stop Sign Controlled Approaches”.

5.1.2 Sight Distances for Undercrossings

While not frequently problems, structure fascias may cut the lines of sight on roads passing under bridges and limit the sight distances to less than what are otherwise attainable. It is generally practical to provide the minimum lengths of sag vertical curves at grade separation structures. Even where the recommended grades are exceeded, the sight distances should not be reduced below the minimum recommended values for sight distances. See [pages 3-177 thru 3-180 of 2018 GDHS](#) for more information.

5.1.3 Passing Sight Distances

Passing sight distance is the minimum sight distance that must be available to enable the driver of one vehicle to pass another vehicle safely and comfortably, without interfering with the speed of an oncoming vehicle traveling at the design speed should it come into view after the overtaking maneuver is started. The sight distances available for passing at any locations are the longest distances at which drivers, whose eyes are 3.5 feet above the pavement surfaces, can see objects 3.5 feet high on the roads (see [Attachment 5.6](#) and [2018 GDHS, pages 3-169 thru 3-172](#) for additional information).

Minimum passing sight distances (found in [Attachments 5.1](#) and [Table 3-4, Page 3-11, 2018 GDHS](#)) are sufficient for single or isolated passing only, and often opposing vehicles will cancel their passing opportunities. It is important to consider adequate passing sight distances over as much of the highway lengths as feasible. The greater the volumes of traffic on the roadways the more important it is to maximize well distributed passing opportunities.

When 'Modernizing' existing facilities, it is important to consider trying to achieve passing opportunities of 60 percent or greater, if possible. It may be more advantageous to flatten smaller vertical curves rather than flattening single large vertical curves.

Guidance on establishing, marking and signing no-passing zones can be found in the Traffic Engineering Operations and Safety Manual (TEOpS) ⁷ and is specified in the [Wisconsin Standard Specifications for Highway and Structure Construction](#). The sight distance values shown are not to be used directly in design but should be reviewed so that locations requiring no-passing zone markings can be recognized during design. Review proposed alignments with the region traffic section staff to assure that the passing sight distances provided will not require no-passing markings.

Any highways (including bypasses and expressways) that are designed as future 4-lane divided highways, but designed to open initially as 2-lane highways, should be designed considering passing sight distance criteria for 2-lane highways. The designs should provide adequate passing sight distances as 2-lane highways for their full

⁶ See Wang et al in Transportation Research Record 1937 Normal Deceleration Behavior of Passenger Vehicles at Stop Sign-Controlled Intersections Evaluated with In-Vehicle Global Positioning System Data. In No. 1937: *Transportation Research Board of the National Academies*, 2005, pp.120-127.

<http://trb.metapress.com/content/u787613013238870/fulltext.pdf>

see also *A Policy on Geometric Design of Highways and Streets*, 7th edition. AASHTO, 2018, [Figure 2-34, Page 2-88](#))

⁷ Applications / No Passing Zone Standards. In [TEOpS 3-2-2, ch. 3: Markings](#) 2009, sect. 3-2-2, pp.1-5.

performance life spans⁸.

5.1.4 Intersection Sight Distances (ISDs), Sight Triangles, Vision Triangles, and Vision Corners

Intersection Sight Distance is the distance for which there must be unobstructed horizontal and vertical sight along both roads of an intersection, and across their included corners that is sufficient to allow the operators of vehicles approaching the intersection or stopped at the intersection, and to safely carry out whatever maneuvers may be required to negotiate the intersection. Intersection Sight Distances are important to evaluate at all at-grade intersections on all projects for both passenger cars and for the design vehicles shown in [Table 5.1](#). Intersection sight distances are ensured by establishing clear sight triangles (see Figure 5.4 and [page 9-37 of 2018 GDHS](#)) across each of the included corners of intersections. Design guidance on intersection sight distances and sight triangles for non-roundabout intersections can be found later in this section and is based on [pages 9-35 thru 9-59 of the 2018 GDHS](#) but modified as noted. Guidance on intersection sight distances for roundabouts can be found in [FDM 11-26-30](#). Guidance for intersection sight distances on Perpetuation and Rehabilitation projects can be found in [FDM 11-40-1](#). See [FDM 11-46-20](#) for guidance on sight distances for trail crossings.

Table 5.1 Design Vehicles for Intersection Sight Distances ^A

| Type of Intersecting Highway | Design Vehicle for Purposes of ISD ^B |
|-----------------------------------|---|
| Interchange ramp terminals | Combination Truck |
| Arterials | Combination Truck |
| Collectors | Single Unit Truck (SU-vehicle) ^C |
| Local Roads / Residential Streets | Single Unit Truck (SU-vehicle) ^C |

A See [FDM 11-25-2.1](#) for guidance on Intersection Design Vehicles and Intersections. Check Vehicles for turning movements at intersections

B Only the Passenger vehicles need be considered in areas where truck traffic is minimal (<2.5% of AADTs), and right-of-way restrictions prohibit adequate sight window clearing

C If there is significant Combination Truck traffic then use those as the design vehicles instead of the Single Unit Trucks.

5.1.4.1 Design Guidance for Intersection Sight Distances

As mentioned above, intersection sight distances are important to evaluate at all at-grade intersections on all projects for both passenger cars and for the design vehicles shown in [Table 5.1](#). Guidance is provided below for the following intersection control cases.

- Case A - Intersections with no control
- Case B - Intersections with stop control on the minor road
 - B1 - Left turn from the minor road
 - B2 - Right turn from the minor road
 - B3 - Crossing maneuver from the minor road
- Case C - Intersections with yield control on the minor road
 - C1 - Crossing maneuver from the minor road
 - C2 - Left or right turn from the minor road
- Case D - Intersections with traffic signal control
- Case E - Intersections with all-way stop control
- Case F - Left turns from major roads
- Case G - Roundabouts

⁸ Review of Wisconsin Bypass Road Design Practices February 13-14, 2006. (Final). Federal Highway Administration Resource Center, 2006. <http://www.dot.wisconsin.gov/library/publications/docs/wis-bypass-report.pdf>

5.1.4.1.1 Case A - Intersections with No Controls

Do not allow uncontrolled at-grade intersections on STHs. For non-STH locations, use guidance from [2018 GDHS, pages 9-40 thru 9-42](#).

5.1.4.1.2 Case B - Intersections with Stop Control on Minor Roads

Gap Acceptance, as described on page 9-43, 2018 GDHS and modified herein, is the basis for computing Case B Intersection Sight Distances.

Decision point locations for side road vehicles are the positions of the side road driver's eyes in relation to the mainlines. For rural intersections, these distances approximate the locations of passenger cars at standard stop bar installations. They are equal to 14.50 feet from the farthest outside edges of non-shoulder mainline pavements. In most situations, these would be either the edges of the mainline right turn lanes, the mainline right-turn tapers, the mainline downstream acceleration tapers, or the mainline travel lanes. However, if there are separate channelized right-turn lanes on the mainlines, these distances would be to either the mainline downstream acceleration tapers, or the edges of the near mainline travel lanes.

For urban intersections, these distances are 14.50 feet from the near faces of mainline curbs.

Decision Point Locations for crossing vehicles stopped in medians are the positions of the driver's eyes in relation to the far side traffic lanes. For medians without stop bars, assume the vehicles stop with their front ends 3 feet from the median edges of travel lanes. The drivers' eyes are 8 feet behind these, or 11 feet from the median edges of travel lanes. For medians with stop bars this distance is about 8.0 feet behind the stop bars.

Median widths need to be at least 6 feet greater than vehicle lengths for vehicles to complete crossings in two (2) steps.⁹

Decision point locations for mainline vehicles vary by design speeds and design vehicles. [Table 5.2](#) shows the Intersection Sight Distance WisDOT requirements for [2018 GDHS Intersection Control Cases B1, B2, and B3](#). See [Attachment 5.11](#) for an example computation.

⁹ A Policy on Geometric Design of Highways and Streets, 7th edition. AASHTO, 2018, [pp 7-17 thru 7-18](#)

Table 5.2 Intersection Sight Distance ^A Criteria for Intersection Control Cases B1, B2, and B3 - Stop on Minor Road¹⁰

| Design vehicle | Case B1 - Left turn from the minor road ^B | | | Case B2 - Right turn from the minor road ^C | | | Case B3 - Crossing maneuver from the minor road ^D | | |
|---------------------------------------|--|----------------------------|----------------------------|---|----------------------------|----------------------------|--|----------------------------|----------------------------|
| | P | SU | Combination Truck | P | SU | Combination Truck | P | SU | Combination Truck |
| Eye height (feet) | 3.5 | 7.6 | 7.6 | 3.5 | 7.6 | 7.6 | 3.5 | 7.6 | 7.6 |
| Time gap (sec) | | | | | | | | | |
| UPPER ¹¹ MINIMUM (MINIMUM) | 10.0 (7.5) | 12.0 (9.5) | 13.0 (11.5) | 8.0 (6.5) | 10.0 (8.5) | 12.0 (10.5) | 7.0 (6.5) | 10.0 (8.5) | 13.0 (10.5) |
| Mainline Design Speed (mph) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) |
| 25 | 370 (280) | 445 (350) | 480 (425) | 295 (240) | 370 (315) | 445 (390) | 260 (240) | 370 (315) | 480 (390) |
| 30 | 445 (335) | 530 (420) | 575 (510) | 355 (290) | 445 (375) | 530 (465) | 310 (290) | 445 (375) | 575 (465) |
| 35 | 515 (390) | 620 (490) | 670 (595) | 415 (335) | 515 (440) | 620 (545) | 365 (335) | 515 (440) | 670 (545) |
| 40 | 590 (445) | 710 (560) | 765 (680) | 475 (385) | 590 (500) | 710 (620) | 415 (385) | 590 (500) | 765 (620) |
| 45 | 665 (500) | 795 (630) | 860 (765) | 530 (430) | 665 (565) | 795 (695) | 465 (430) | 665 (565) | 860 (695) |
| 50 | 735 (555) | 885 (700) | 960 (850) | 590 (480) | 735 (625) | 885 (775) | 515 (480) | 735 (625) | 960 (775) |
| 55 | 810 (610) | 975 (770) | 1055 (930) | 650 (530) | 810 (690) | 975 (850) | 570 (530) | 810 (690) | 1055 (850) |
| 60 | 885 (665) | 1060 (840) | 1150 (1015) | 710 (575) | 885 (750) | 1060 (930) | 620 (575) | 885 (750) | 1150 (930) |
| 65 | 960 (720) | 1150 (910) | 1245 (1100) | 765 (625) | 960 (815) | 1150 (1005) | 670 (625) | 960 (815) | 1245 (1005) |
| 70 | 1030 (775) | 1235 (980) | 1340 (1185) | 825 (670) | 1030 (875) | 1235 (1085) | 725 (670) | 1030 (875) | 1340 (1085) |

¹⁰ NCHRP Report 383: Intersection Sight Distance. TRB, National Research Council, 1996. Intersections at Grade. In FHWA-RD-01-051: Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians Federal Highway Administration Turner-Fairbank Research Center, 2001, Section I. https://safety.fhwa.dot.gov/intersection/other_topics/fhwasa09027/resources/Guidelines%20and%20Recommendations%20to%20Accommodate%20Older%20Drivers.pdf.

¹¹ Upper minimum time gap was derived through a study done by WisDOT and documented in the file named: ISD and vision triangle recomm vs 1990 GDHS.xls. Wisconsin DOT, 2004.

- A Intersection Sight Distance formula is found on [Page 9-45, 2018 GDHS](#). Table values have been rounded.
- B Case B1 Time gaps and intersection sight distances are for stopped vehicles to turn left onto two-lane highways with no medians and grades 3 percent or less. The table values require adjustment as follows:
- For multilane highways. For left turns onto two-way highways with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, from the left in excess of one, to be crossed by the turning vehicles. Medians are computed as equivalent lane widths if they are too narrow for vehicles to stop in, e.g. a 30-foot median would be equivalent to 2.5 lanes. Mainline right turn lanes and tapers are also treated as equivalent lane widths.
 - For minor road approach grades. If the approach grades are upgrades that exceed 3 percent, then add 0.2 seconds for each percent grade for left turns.
 - For skews, use guidance from [pages 9-58 thru 9-59, 2018 GDHS](#).
- C Case B2 Time gaps and intersection sight distances are for stopped vehicles to turn right onto two-lane highways with grades 3 percent or less. The table values require adjustment as follows:
- Add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, from the left in excess of zero, to be crossed by the turning vehicles. Mainline right turn lanes and tapers are treated as equivalent lane widths.
 - For minor road approach grades. If the approach grades are upgrades that exceeds 3 percent then add 0.1 seconds for each percent grade.
 - For skews, use guidance from [pages 9-58 thru 9-59, 2018 GDHS](#).
- D Case B3 Time gaps and intersection sight distances are for stopped vehicles to cross two-lane highways with no medians and grades 3 percent or less. The table values require adjustment as follows:
- For multilane highways. For crossing major roads with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane to be crossed and for narrow medians that cannot store the design vehicles. Medians are computed as equivalent lane widths if they are too narrow for vehicles to stop in, e.g. 30-foot medians would be equivalent to 2.5 lanes. Mainline right turn lanes and tapers are also treated as equivalent lane widths.
 - For minor road approach grades. If the approach grades are upgrades that exceed 3 percent, then add 0.2 seconds for each percent grade.
 - For skews, use guidance from [pages 9-58 thru 9-59, 2018 GDHS](#).

5.1.4.1.3 Case C - Intersections with Yield Control on the Minor Roads

Except for roundabouts, do not allow yield-controlled at-grade intersections on STHs. Use the guidance from [pages 9-50 thru 9-55, 2018 GDHS](#) for non-roundabout yield-controlled intersections on non-STH roads - except use Case B1 and B2 values, per [Table 5.2](#), for mainline ISD distances for Case C2 - Left or right turns from the minor roads (see [Section 9.5.3.3.2, page 9-53, 2018 GDHS](#)).

5.1.4.1.4 Case D - Intersections with Traffic Signal Controls

Use the guidance from [pages 9-55 thru 9-56, 2018 GDHS](#), except use the values from [Table 5.2](#) where Case B is called for, e.g. where right turns on red are allowed.

5.1.4.1.5 Case E - Intersections with All-Way Stop Control (AWSC)

At intersections with all-way stop controls, the first stopped vehicles on one approach should be visible to the drivers of the first stopped vehicles on each of the other approaches. There are no other sight distance criteria applicable to intersections with all-way stop controls. See [page 9-56, 2018 GDHS](#)¹².

5.1.4.1.6 Case F - Left Turns from the Major Roads

Use the guidance from [pages 9-56 thru 9-58, 2018 GDHS](#), except use the time gaps and Intersection Sight Distances shown in [Table 5.3](#).

Provide positive offsets for opposing left turn lanes, if possible. Positive left-turn lane offsets, as shown in [Figure 5.1](#), can be helpful in allowing drivers in opposing left-turn bays to see past each other to detect oncoming traffic. They can also improve the operations of signalized intersections by allowing more efficient use of permissive left-turn phasing. See [FDM 11-25-5.4.1](#), "Slotted Left-turn Lanes" for additional guidance.

¹² A Policy on Geometric Design of Highways and Streets, 7th edition. AASHTO, 2018.

Table 5.3 Intersection Sight Distance (ISD) Criteria for Case F - Left Turn from Major Road¹³

| Design vehicle | P | SU | Combination Truck |
|---|----------------------------------|----------------------------------|----------------------------------|
| Eye height (ft) | 3.5 | 7.6 | 7.6 |
| Time gap ^A (sec) UPPER MINIMUM ¹⁴ (MINIMUM) | 8.0 (5.5) | 8.0 (6.5) | 8.0 (7.5) |
| Mainline Design Speed (mph) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) | ISD (feet) UPPER MIN (MIN) |
| 25 | 295 (205) | 295 (240) | 295 (280) |
| 30 | 355 (245) | 355 (290) | 355 (335) |
| 35 | 415 (285) | 415 (335) | 415 (390) |
| 40 | 475 (325) | 475 (385) | 475 (445) |
| 45 | 530 (365) | 530 (430) | 530 (500) |
| 50 | 590 (405) | 590 (480) | 590 (555) |
| 55 | 650 (445) | 650 (530) | 650 (610) |
| 60 | 710 (490) | 710 (575) | 710 (665) |
| 65 | 765 (530) | 765 (625) | 765 (720) |
| 70 | 825 (570) | 825 (670) | 825 (775) |

^A Time gaps and intersection sight distances are for vehicles making left turns from undivided 2-lane highways (1 lane in each direction). The table values require adjustment as follows: For left-turning vehicles that cross more than one opposing lane, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane to be crossed. Median widths crossed are computed as equivalent lane widths and are measured from the inside edges of left turn lanes to the median edges of the opposing travel lanes. See [page 9-57, 2018 GDHS](#).

¹³ Intersections at Grade. In *FHWA-RD-01-051: Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians* Federal Highway Administration Turner-Fairbank Research Center, 2001, Section I. https://safety.fhwa.dot.gov/intersection/other_topics/fhwas09027/resources/Guidelines%20and%20Recommendations%20to%20Accommodate%20Older%20Drivers.pdf

¹⁴ Upper minimum time gap was derived through a study done by WisDOT and documented in the file named: *ISD and vision triangle recomm vs 1990 GDHS.xls*. Wisconsin DOT, 2004.

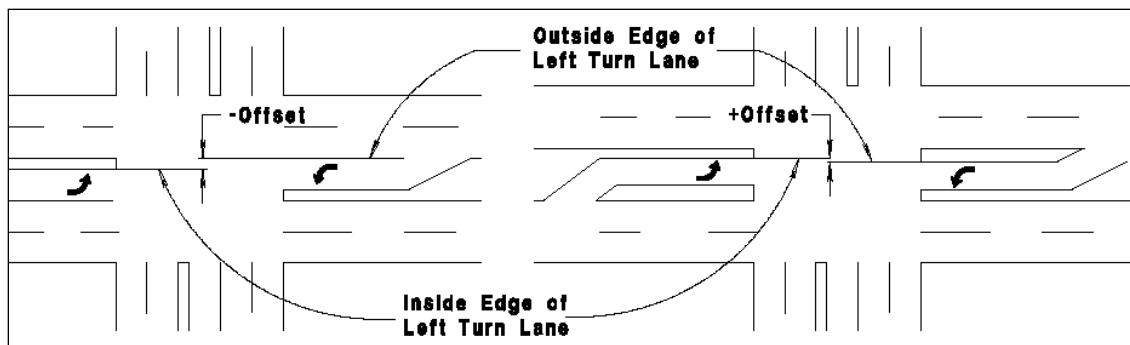


Figure 5.1 Negative (-) Offset and Positive (+) Offset of Opposing Left Turn Lanes¹⁵

5.1.4.1.7 Case G - Roundabouts

Use the guidance from [page 9-58, 2018 GDHS](#), and [FDM 11-26](#).

5.1.4.1.8 Interchange Ramp Terminals at Crossroads

Treat ramp terminals at crossroads like any other at-grade intersections. Provide intersection sight distances based on the applicable intersection control case as described above. However, be very conscious of potential sight obstructions such as bridge railings, piers, and abutments that are likely to be found near ramp terminals. Design the crossroad profiles, and also provide enough separation between the ramp terminal intersections and the structures, so that intersection sight distances are not obstructed.

If the crossroads are under-crossings, check the sight distances under the structures graphically using the appropriate eye heights for the vehicles at the intersections, and object heights of at least 2.0 feet.

Traffic controls such as signals, all-way stop signs, or roundabouts may be possible solutions for ramp terminal locations where there are inadequate sight distances because less intersection sight distances are needed.

5.1.4.1.9 Mitigation Measures for Sight Distance Deficiencies at Intersections

Some intersections may have either inadequate intersection sight distances or inadequate roadway sight distances approaching the intersections that are causing safety and operational problems. Ideally, the deficiencies would be corrected in timely and cost-effective manners. However, this is not always possible. Consult with the Region Traffic Section on possible mitigation measures. Some mitigation measures that might be considered-either alone or in combination-are:

- Restrict or prohibit some turning movements.
- Reduce the regulatory speeds. Use appropriate signing and warning lights.
- All-way stop sign controls, traffic signals, or roundabouts - these require that there be adequate sight distances on the roadways approaching the intersections.
- Provide travel lane rumble strips (typically for approaching stop signs). Be aware of noise and proximity to houses.
- Provide advance signing, and possibly warning lights.
- Adjust signing at intersections so that they are visible from farther away. Do this by making the signs larger, brighter, or providing additional signs and marking, or a combination of any of the three.
- Provide pork chop islands on side-road approaches to allow for more effective placement of supplemental STOP signs or traffic signals, and to encourage better positioning of stopped vehicles for enhanced visibility of approaching mainline traffic.
- Provide street lighting at the intersections. Light poles also provide daytime recognition of the presence of intersections.
- Use "CROSS TRAFFIC DOES NOT STOP" signs at intersections that drivers could misinterpret as all-way stops (for example, two roads of equal status intersect and only one of them has a stop sign).

¹⁵ *Median Handbook*. Florida DOT, 2001, Ch 3, p8

See the following references for additional safety measures and mitigation¹⁶:

- NCHRP Report 500, vol. 5 and vol. 12,
- FHWA bypass report,
- FHWA-SA-09-020, “Low-Cost Safety Enhancements for Stop-Controlled and Signalized Intersections”

5.1.4.1.10 Sight Distances for Railroad-Highway Grade Crossings

See [FDM Chapter 17](#) Railroad Coordination.

5.1.4.2 Clear Sight Triangles

A clear sight triangle and its horizontal and vertical boundaries are shown in [Figure 5.2](#). The dimensions of clear sight triangle vary depending on the types of vehicles and the types of intersection controls (see [Table 5.1](#) above and Design Guidance for Intersection Sight Distances below).

In establishing sight lines through clear sight triangles, use eye heights above the roadway surfaces of 3.5 feet for passenger cars and 7.6 feet for trucks. Use object heights above the roadway surfaces of 3.5 feet.

¹⁶ (22) *NCHRP Report 500: A Guide for Addressing Unsignalized Intersection Collisions*, vol 5 of Guidance for Implementation of the AASHTO Strategic Highway Safety Plan. Transportation Research Board of the National Academies, 2003. http://onlinepubs.trb.org/Onlinepubs/nchrp/nchrp_rpt_500v5.pdf

(23) *NCHRP Report 500: A Guide for Reducing Collisions at Signalized Intersections*, vol 12 of Guidance for Implementation of the AASHTO Strategic Highway Safety Plan. Transportation Research Board of the National Academies, 2004. http://onlinepubs.trb.org/Onlinepubs/nchrp/nchrp_rpt_500v12.pdf

(10) *Review of Wisconsin Bypass Road Design Practices February 13-14, 2006. (Final)*. Federal Highway Administration Resource Center, 2006. <http://www.dot.wisconsin.gov/library/publications/docs/wis-bypass-report.pdf>

(24) *Low-Cost Safety Enhancements for Stop-Controlled and Signalized Intersections*. FHWA-SA-09-020. Federal Highway Administration, 2009. <http://safety.fhwa.dot.gov/intersection/resources/fhwasa09020/fhwasa09020.pdf>

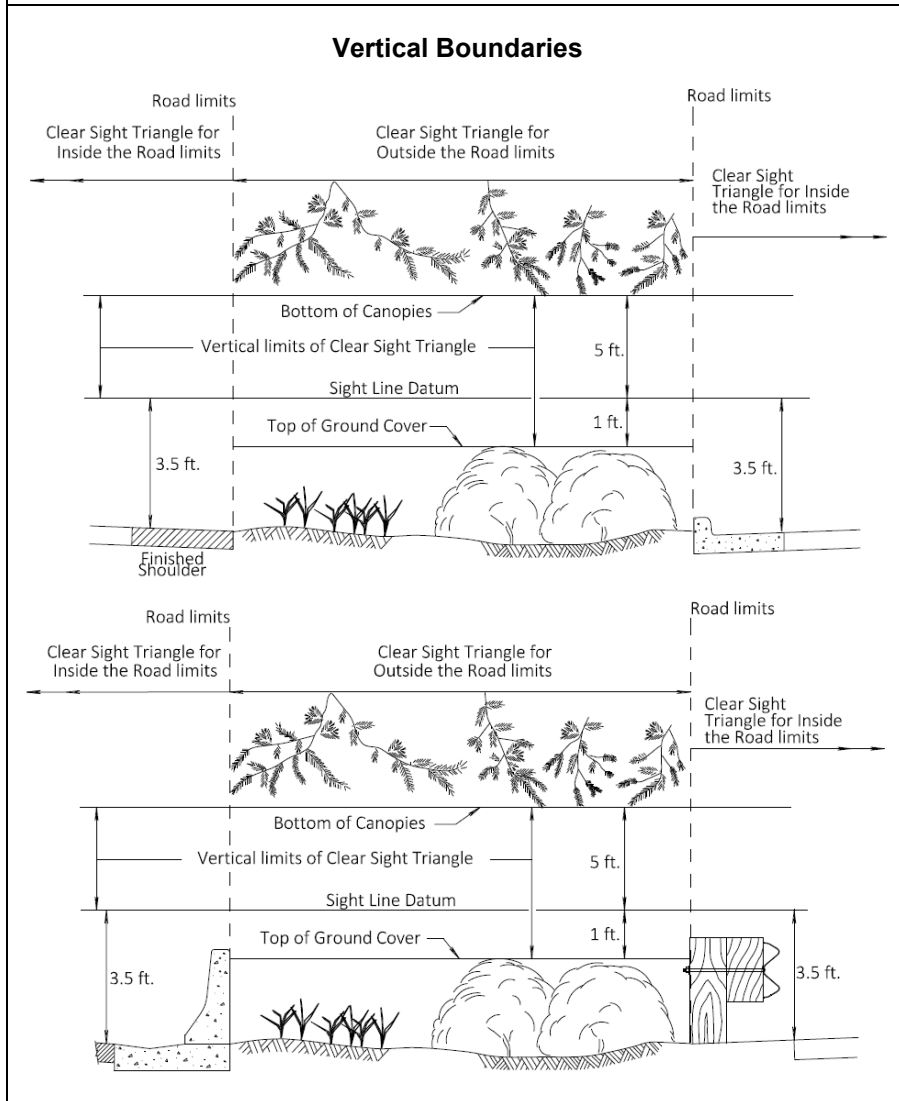
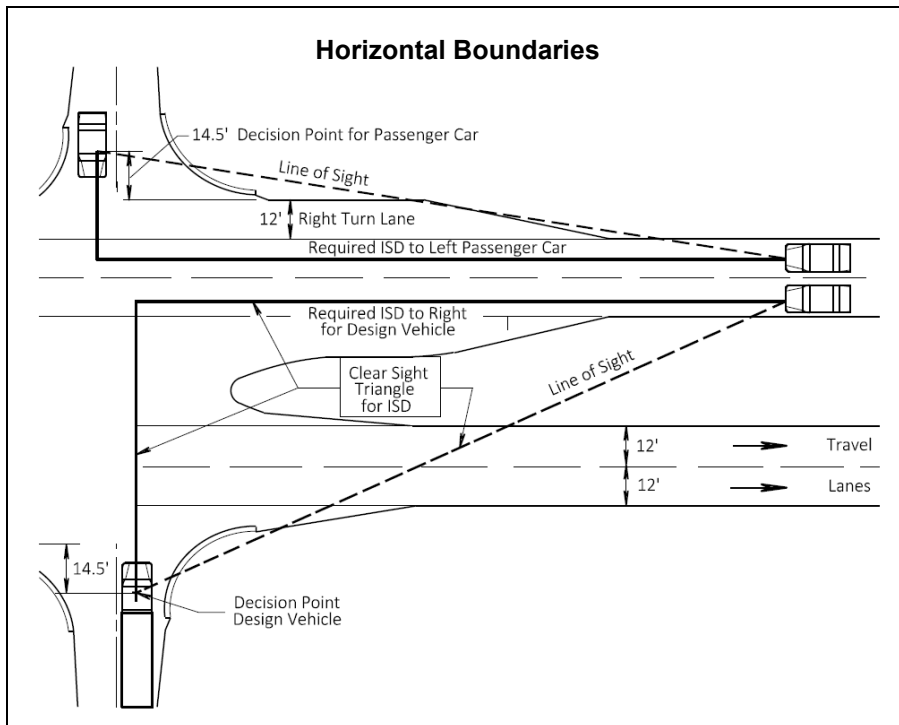


Figure 5.2 Clear Sight Triangles¹⁷

5.1.4.2.1 Horizontal Boundaries of Clear Sight Triangles

The horizontal boundaries of clear sight triangles on the mainlines are the centers of the approach travel lanes, beginning at the intersections and ending at points known as the decision points for the mainlines, established by applying the intersection sight distance requirements for the intersection control cases (see [FDM 11-10-5.1.4.2](#) “Design Guidance for Intersection Sight Distances” below).

The horizontal boundaries of clear sight triangles on the side roads is the centers of the approach travel lanes, beginning at the intersections and ending at points known as the decision points for the side roads established by applying the intersection sight distances for the intersection control cases (see [FDM 11-10-5.1.4.2](#) “Design Guidance for Intersection Sight Distances” below).

The horizontal boundary of clear sight triangles across the included corners of intersections are the lines connecting the end points, or decision points, of the first two sides.

5.1.4.2.2 Vertical Boundaries of Clear Sight Triangles

The bottom boundaries of clear sight triangles are the sight line datums inside the road limits, and 1-foot below the sight line datums outside the road limits. A sight line datum is defined as a line of sight which is 3.5-feet above the pavement surface on each end. The road limit is defined as the edge of finished shoulder or the back of curb, whichever is applicable, unless there is a barrier or bridge parapet. In those cases, the road limits are defined as the back edges of roadside barriers or bridge parapets.

The top boundaries of clear sight triangles are 5-foot above the sight line datums.

5.1.4.2.3 Obstructions Within Clear Sight Triangles

Make sure that clear sight triangles are clear of obstructions that might block drivers' views of potentially conflicting vehicles. These include, but are not limited to:

- The roadways themselves - check the vertical alignments and super elevations of the highways to see if the pavements obscure the lines of sight,
- Roadside and median barriers - including beam guard,
- Bridge parapets and railings,
- Cut slopes and embankments,
- On-street parked vehicles - See [Figure 5.3](#). Consider prohibiting on-street parking as follows, unless greater restrictions are required by statute, the Wisconsin MUTCD, or to provide adequate lines of sight for pedestrians:
 - **Upper Minimum:** within the Intersection Sight Distance clear sight triangles¹⁸.
 - **Minimum:** so that vehicles entering from the side-roads do not have to encroach on the mainline travel lanes or bicycle lanes in order for the drivers to see vehicles approaching on the mainlines at a distance equal to ISD.
 - **NOTE:** assume parked vehicles are 12 inches from the curb face.
- Off-street parked vehicles - Consider prohibiting off-street parking within the ISD clear sight triangles,
- Signal control cabinets,
- Landscaping,
- Signs - offset signs to prevent sight distance obstructions,
- Structures, including, but not limited to, buildings, fences, retaining walls, screenings,
- Vegetation, including bushes, hedges, natural growths, plantings, tall crops, tree branches, and tree trunks.

There must be sufficient right-of-way to ensure that line-of-sight obstructions can be removed.

¹⁷ *A Policy on Geometric Design of Highways and Streets*, 7th edition. AASHTO, 2018, [Section 9.5.2](#)

¹⁸ *Access Management Manual*. Transportation Research Board, 2003., Figure 5.8-5.9

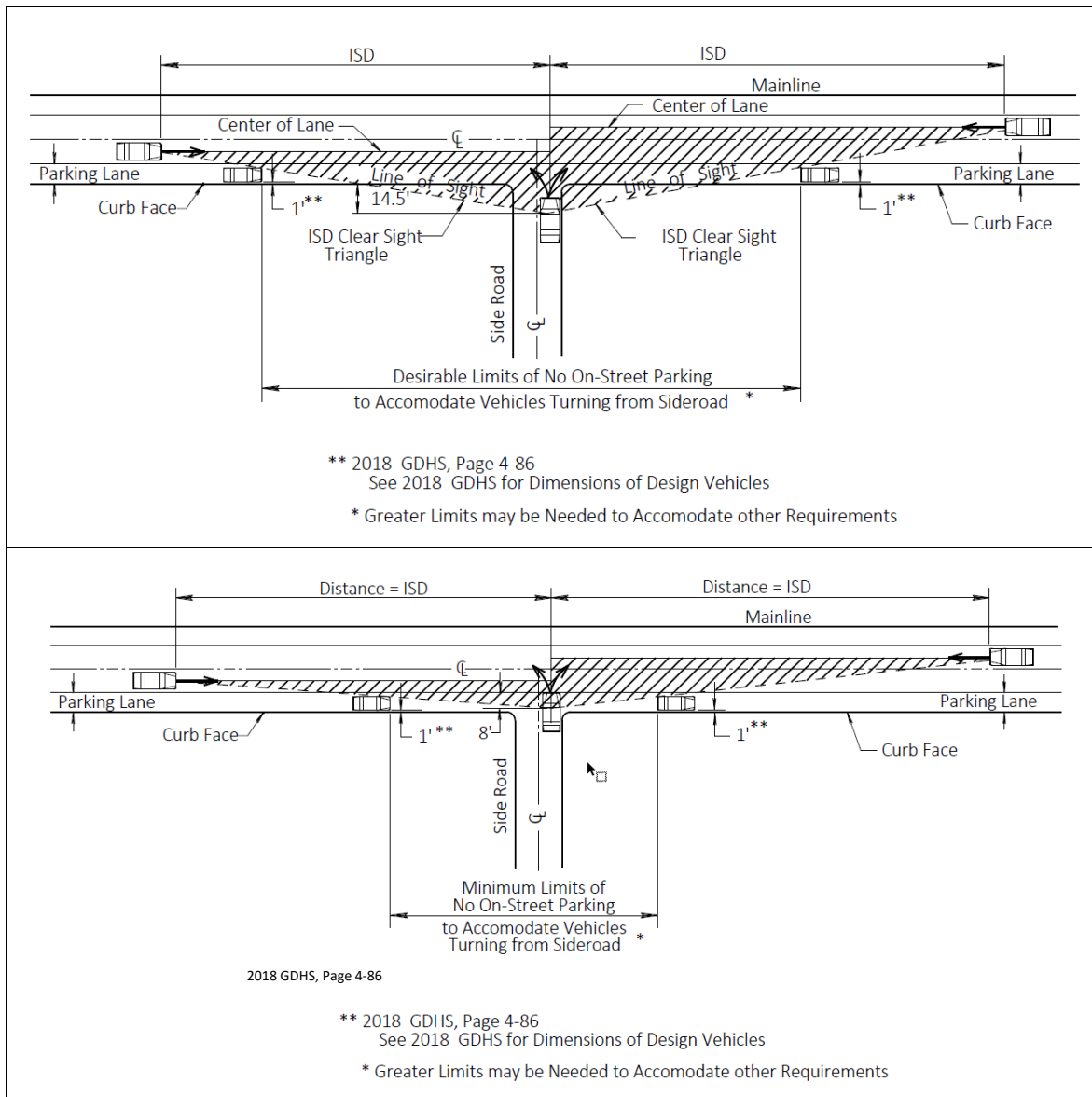


Figure 5.3 Determining On-Street Parking Limits¹⁹

Street markers, traffic signs, and other traffic controls are allowed within the clear sight triangles, provided their numbers and arrangements do not significantly block vision across the areas. Likewise, utility pedestals and poles are allowed within the clear sight triangles, provided their numbers and arrangements do not significantly block vision across the areas. Consider offsetting right turn lanes at intersections where there are significant numbers of right turns that impede clear sight lines.

Urban intersections can be particularly concerning because of the abundance of street furniture and developments in the vicinities of intersections. Pay particular attention to the potential for line-of-sight obstructions in the placement of signs, light poles, signal controllers, tree plantings, newspaper, and advertising boxes, etc.

Pedestrian considerations: Features such as landscaping, parked cars, utility poles, traffic control devices, and street furniture can create sight obstructions for pedestrians. Consider installing curb extensions or instituting parking restrictions to ensure that pedestrian sight lines are not blocked.

¹⁹ **Note:** This is not an alternative method for establishing ISD. Parked cars are not permanent sight obstructions. ISD from 14.50 feet behind the near face of mainline curb is still important to evaluate per [FDM 11-10-5.1.4.2.2](#).

5.1.4.3 Design Guidance for Vision Triangles²⁰

Where there are stop signs or traffic signal controls on the side roads, the clear sight triangles for intersection sight distances extend only short distances along the side roads because it is assumed that the drivers of vehicles approaching on the side roads will see and obey the traffic controls. However, there are millions of crossing path crashes every year in this country caused by drivers running stop signs and red lights²¹. Adding vision triangles enhances the safety of intersections by providing opportunities for speed adjustments or evasive maneuvers by vehicles on the major highways in the events vehicles on the minor roads violate the traffic controls. These safety enhancements can be particularly important at higher volume intersections on high-speed roads.

A vision triangle is an additional area clear of obstructions that might block a driver's view of potentially conflicting vehicles, for intersections with stop sign control on the side road and for signal-controlled intersections. Their purpose is to provide opportunities for speed adjustments or evasive maneuvers by vehicles on the major highways if vehicles on the minor roads violate the traffic control. ISD's and sight triangles should be provided at all intersections whether or not vision triangles are provided.

As mentioned in the previous paragraph, vision triangles are additional clear sight triangles for intersections with stop sign controls on the side roads and for signal-controlled intersections. In other words, vision triangles are supplements to, and not substitutes for, Intersection Sight Distances (ISDs) and sight triangles (see [9-35 thru 9-59 of the 2018 GDHS](#)).

Guide dimensions for vision triangles are shown on [Attachment 5.10](#). The dimensions are based on providing two seconds of travel time at the posted speeds+5 mph for both the mainlines and the side roads, i.e., they are reciprocal with respect to the time both drivers can see and react to each other. Greater dimensions may be used if desired - e.g., if local zoning ordinances show greater distances. On the other hand, if site conditions, such as building takings or unacceptable environmental impacts, preclude obtaining the recommended triangles, partial vision triangles can still be beneficial.

Do not rely on vision triangles as the sole protection against run-the-stop-sign crashes. Measures such as increasing visibility of traffic control devices and providing streetlights can aid approaching drivers in detecting intersections and their controls.

Modernization projects should evaluate providing vision triangles per the criteria shown below. If it is not possible to provide vision triangles, justifications are required in the Design Study Reports (DSRs). Vision triangles are not used at roundabouts because research has shown that excessive intersection sight distances at roundabouts result in higher frequencies of crashes.

5.1.4.3.1 Criteria for Providing Vision Triangles

For Modernization Projects:

- DO NOT provide vision triangles at roundabouts
- Evaluate at at-grade intersections on expressways.
- Evaluate at other intersections and at driveways which meet either of the following warrants:
 - the posted speeds of the STHs ≥ 45 mph, and current or construction year traffic volumes on the STHs > 750 AADT, and current or construction year traffic volumes on the side roads (or driveways) > 400 AADT, and the sum of both > 1250 AADT.
 - the posted speeds of the STHs ≥ 45 mph, and design year traffic volumes on the STHs > 2500 AADT and design year traffic volumes on the side roads (or driveways) > 1000 AADT.
- Provide at intersections where there has been a history of run-the-stop sign or run the red-light crashes.
- Perpetuate at intersections where they have been previously provided.
- Vision triangles are optional at other intersections and driveways.

For Rehabilitation Projects:

- DO NOT provide vision triangles at roundabouts

²⁰ A study done by WisDOT and documented in the file named: *ISD and vision triangle recomb vs 1990 GDHS.xls*. Wisconsin DOT, 2004.

²¹ References: *Intersection Collision Avoidance Using ITS Countermeasures*. NHTSA, 2000. <https://rosap.ntl.bts.gov/view/dot/14250>, p6.10; *Intersection Decision Support Project: Taxonomy of Crossing-Path Crashes at Intersections Using GES 2000 Data*. University of California Traffic Safety Center, 2003. <http://repositories.cdlib.org/cgi/viewcontent.cgi?article=1006&context=its/tsc>

- Evaluate at intersections which are being upgraded, and which meet either of the following warrants:
 - the posted speeds of the STHs ≥ 45 mph, and current or construction year traffic volumes on the STHs > 750 AADT, and current or construction year traffic volumes on the side roads (or driveways) > 400 AADT, and the sum of both > 1250 AADT.
 - the posted speeds of the STHs ≥ 45 mph, and design year traffic volumes on the STHs > 2500 AADT, and design year traffic volumes on the side roads > 1000 AADT.
- Provide at intersections where there has been a history of run-the-stop sign or run-the red-light crashes.
- Perpetuate at intersections where they have been previously provided.
- Vision triangles are optional at other intersections and driveways.

For Perpetuation Projects:

- DO NOT provide vision triangles at roundabouts
- Vision triangles are optional at other intersections and driveways.

5.1.4.3.2 Land Rights and Interests for Vision Triangles

See [FDM 12-1-15](#) for definitions of the various types of land rights and interests acquired by the Department. In order of preference, land rights or interests for vision triangles can be:

1. Fee Titles,
2. Restricted Development Easements - when fee title interests will have significant adverse impacts on the parcels.
3. None - This only applies to vision triangles that are established by local zoning ordinances, since these are not necessarily dedicated as road right-of-way. In these cases, enforcement is through the local zoning authorities.

5.1.4.4 Vision Corners²²

A "vision corner" is defined as either

- The sight triangle for ISD, if no vision triangle is used, or
- The combination of the sight triangle and the vision triangle, as shown in [Figure 5.4](#).

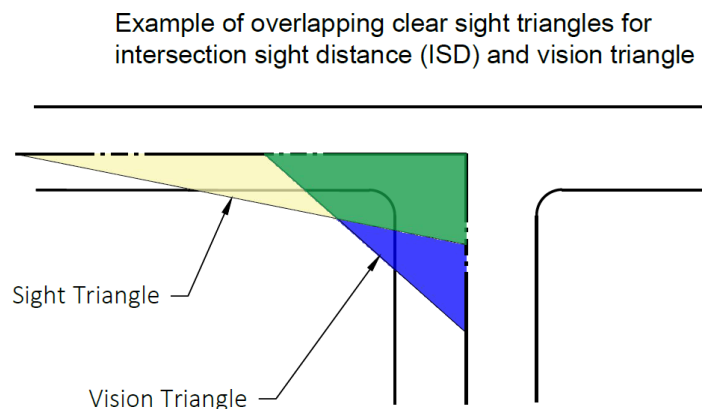


Figure 5.4 Example of Vision Corner

5.2 Horizontal Alignments²³

Horizontal alignments should typically be as straight as possible and consistent with environmental, physical, and economic constraints. Whenever feasible, avoid using maximum curvatures. Flatter curvatures with shorter tangents are generally preferable to sharp curves connected by long tangents. Alignments must be consistent. Sudden changes from flat to sharp curves and long tangents followed by sharp curves can create safety

²² A study done by WisDOT and documented in the file named: *ISD and vision triangle recomb vs 1990 GDHS.xls*. Wisconsin DOT, 2004.

²³ *A Policy on Geometric Design of Highways and Streets*, 7th edition, AASHTO, 2018, [Chapter 3 Elements of Design, Page 3-19, Horizontal Alignment](#)

hazards. Likewise, avoid using reverse curves unless sufficient lengths of tangents are included between the curves to provide for superelevation transitions. Also, avoid using very long curves because they inhibit some drivers from making passing maneuvers even when adequate sight distances exist.

The horizontal alignment development process can possibly introduce trial alignments that have curvatures, superelevations, or superelevation transitions carried onto or through structures. Such alignments should be avoided, except when there are definite needs, or specific purposes. These situations almost always result in unsightly appearances of bridges or bridge railings and create needless complications in design and construction. Safety considerations are paramount however and shall not be sacrificed to meet the foregoing criteria. If the designer feels a bridge must be built on superelevation or transition, the reasons for this should be explained in the Structure Survey Report.

Horizontal curves should not be introduced near the crests of vertical curves. The combination of horizontal and vertical curves can greatly reduce sight distances creating hazardous conditions. These conditions hide the horizontal curves from the approaching drivers, especially at night. These hazards can be avoided by having the horizontal curvatures lead the vertical curvatures; i.e., the horizontal curves are made longer than the vertical curves. Although the designers must attempt to optimize the horizontal alignments with respect to other factors and avoid the appearances of inconsistencies and distortions in the alignments, the horizontal alignments should be coordinated with the vertical and cross-sectional features of the highways.

Adequate sight distances are important to provide on horizontal curves. If objects off the pavements such as bridge piers, cut slopes, or natural growth restricts sight distances then the minimum radii of curvature should be designed considering the sight distances and the lateral clearances to the objects. [Attachment 5.7](#) shows the relationships between obstructions, degrees of curve, design speeds, and sight distances. If horizontal sight distances are not achieved then documentation and approval is needed in the SCDs or DJs in accordance with [FDM 11-38](#), [FDM 11-1-20](#) and [FDM 11-1-4.10](#).

Although the use of P.I.'s without accompanying horizontal curves is discouraged, there may be situations where they are necessary. These are to be discussed and justified in the DSR. [Table 5.4](#) shows maximum deflections without horizontal curves.

[Table 5.5](#) shows the maximum deflections through low-speed urban intersections - both for lane shifts and for centerline deflections. Short curves may be desirable at each end of lane shifts, especially if pavement markings are used through the intersections to provide positive guidance to motorists.

If possible, avoid lane shifts at signalized intersections - particularly where mounting signal heads over each lane. They complicate the designs and may confuse drivers. Also, avoid deflections thru signalized intersections if possible. Also, avoid where lane designation signs are used.

Table 5.4 Maximum Deflections Without Horizontal Curves²⁴

| Posted Speed (S) mph | | Deflection Δ* Lower Maximum | Deflection Δ* Maximum |
|----------------------|----|-----------------------------------|--------------------------|
| Low Speed | 25 | 3° 45' | 5° 30' |
| | 30 | 2° 45' | 3° 45' |
| | 35 | 2° 15' | 2° 45' |
| | 40 | 1° 45' | 2° 15' |
| High Speed | 45 | 1° 15' | 1° 15' |
| | 50 | 1° 00' | 1° 15' |
| | 55 | 1° 00' | 1° 00' |
| | 60 | 0° 45' | 1° 00' |
| | 65 | 0° 45' | 0° 45' |

* Rounded to nearest 15'

Use the Lower Maximum deflection values when appropriate and the Maximum deflection values to MATCH existing conditions or when justified. Providing greater than the Maximum Deflection Values requires approved SCDs or DJs.

Based on the following formulas:

Lower Maximum Deflection Values:

- Low Speed: $TAN \Delta = 60 / (S+5)^2$
- High Speed: $TAN \Delta = 1.0 / (S+5)$

Maximum

- Low Speed: $TAN \Delta = 60 / S^2$
- High Speed: $TAN \Delta = 1.0 / S$

Where:

- S = Posted Speed
- Δ = Deflection Angle

Minimum distances between consecutive horizontal deflections (i.e., P.I.'s) are:

- Low Speed: 100'
- High Speed: 200'

Table 5.5 Maximum Deflections for Through Lanes Through Urban Intersections

| Posted Speed | 25 | 30 | 35 | 40 |
|--------------------|--------|--------|--------|--------|
| Maximum Deflection | 7° 30' | 5° 30' | 4° 15' | 3° 15' |

²⁴Adapted from Horizontal Alignment - Maximum Centerline Deflection without Horizontal Curve. In *OHDOT Location & Design Manual, Vol. 1, Roadway Design ch. 200: Horizontal and Vertical Design* Ohio DOT, 2021, Section 202.2, Fig. 202-1 (July, 2012). and from Temporary Traffic Control Elements – Tapers in *Manual on Uniform Traffic Control Devices Chapter 6: Temporary Traffic Control* Federal Highway Administration, 2012, Section 6C.08, Pages 556-558.

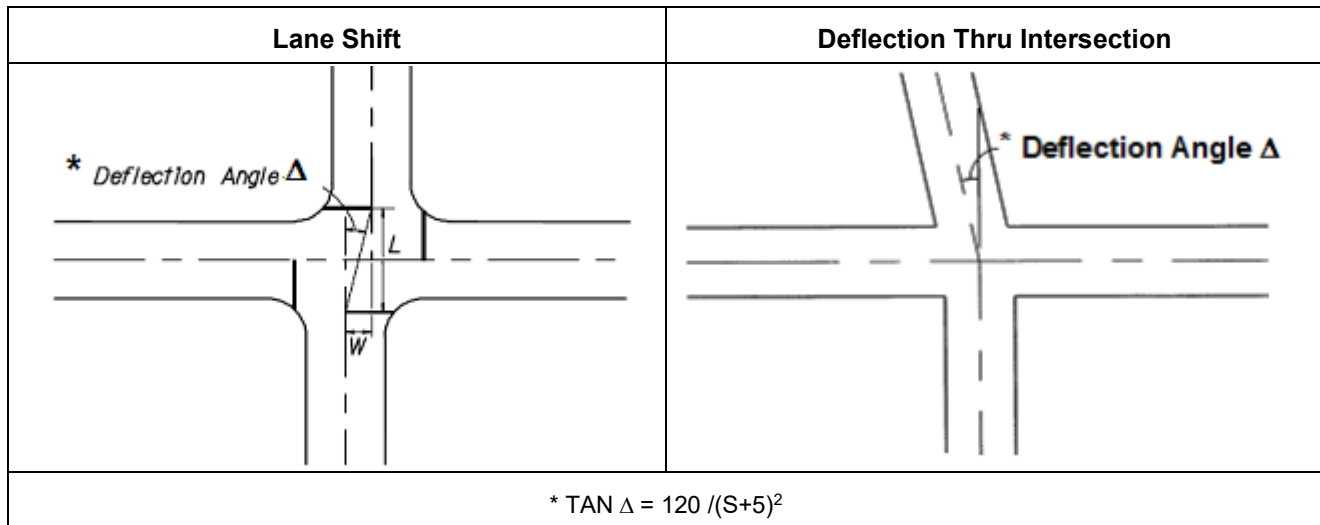


Figure 5.5 Deflection Angle²⁵

5.2.1 Reference Lines

Basic highway reference lines should be the centerlines of normal two-way roadways. Basic reference lines of divided highways may be located either along the centerlines of the medians or along the median edges of the right-hand through pavements in the directions of stationing. All stationing and profiles of finished grades and original ground should be referred to the basic highway reference lines. Auxiliary reference lines along the median edges of left-hand pavements may be desirable when roadways are not parallel or concentric or are widely separated.

Stationing of projects (main lines and side roads) should be from west to east or south to north based on the cardinal directions of the overall highway routes, not just the portion(s) of the highways within the projects under design.

5.2.2 Horizontal Curves on Stop Sign Controlled Approaches

Horizontal curves close to intersections on stop-sign controlled approaches, as shown in [Figure 5.6](#), need to accommodate reasonable operating speeds, while minimizing the potential for adverse operations on super-elevated pavements during snow and ice conditions. Use the following guidelines²⁶:

- Make sure that the stop signs are perceptible to the drivers for sufficient distances from the intersections to allow deceleration before reaching the curves (see [FDM 11-10-5.1.1.4](#), "Sight Distance on Stop Sign Controlled Approaches").
- Assume design speeds for horizontal curves of 20 mph less than the side-road design speeds, but not less than 30 mph if the side-road design speeds are less than or equal to 50 mph.
- Limit the superelevation rates on the approach curves to intersections to 5% or less. The objective is to use as flat alignments as practical with lower superelevations. The preferred designs are to maintain normal crown sections through the curves.
- Provide tangent sections prior to the intersections so that the superelevation runoffs occur outside of the intersection radius returns.

²⁵ Temporary Traffic Control Elements – Tapers in [Manual on Uniform Traffic Control Devices Chapter 6: Temporary Traffic Control](#) Federal Highway Administration, 2012, Section 6C.08, Pages 556-558.

Curves - Horizontal Curves. In [FLDOT Plans Preparation Manual, Volume I - English ch. 2: Design Geometrics and Criteria](#) Florida DOT, 2017, Section 2.8.1, Pages 2-37 thru 2-41.

²⁶ Adapted from [ILDOT Bureau of Design and Environment Manual Chapter 36: Intersections](#). Illinois DOT, 2020.

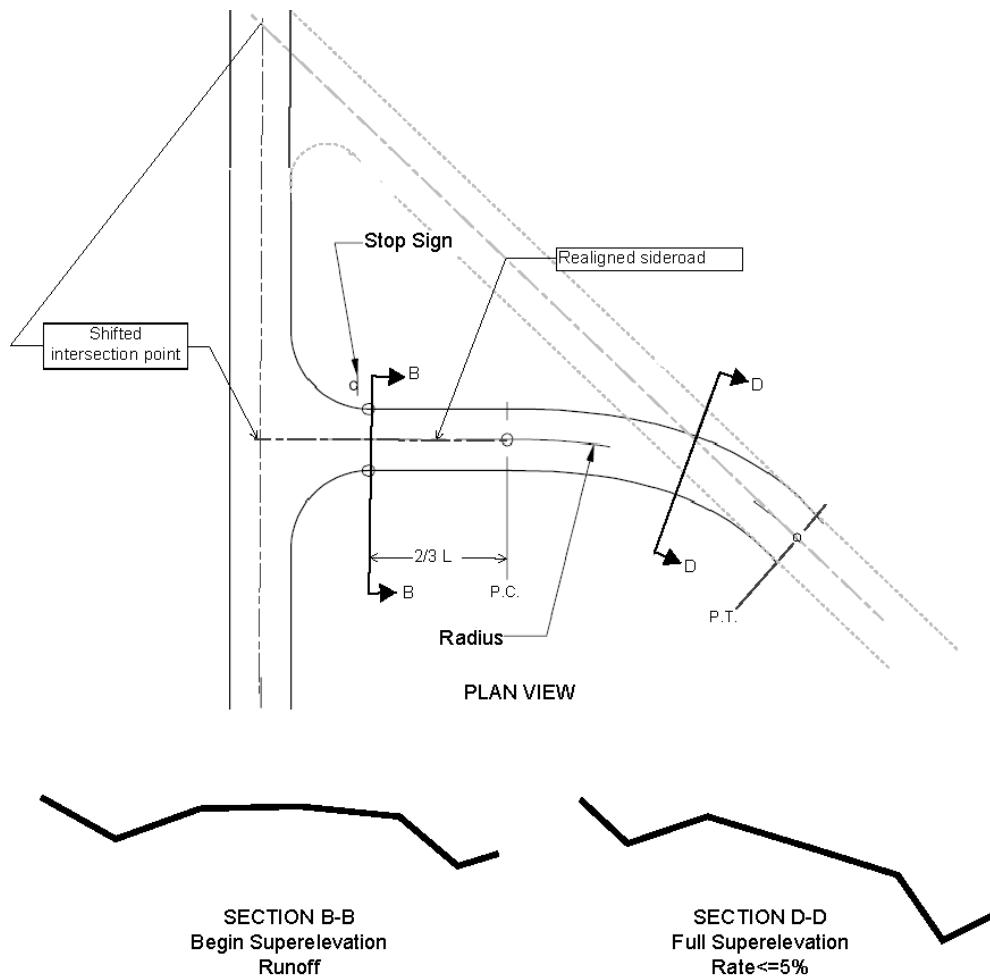


Figure 5.6 Horizontal Curves on Stop Sign Controlled Approaches

5.3 Superelevations

To maintain the desired design speeds, highway and ramp curves are generally super-elevated.

Superelevation may be defined as the rotation of the roadway cross section to overcome part of the centrifugal force that acts on a vehicle traveling around a curve.

Lack of adequate superelevations, where needed, can result in undesirable conditions including: loss of safety factor between the side frictions available versus side frictions used; driver failures to maintain appropriate lateral positioning within the lanes and increased occupant discomfort.

5.3.1 Section intentionally left blank

5.3.2 Superelevation Rates

These rates of rise in cross sections of finished surfaces of the traveled ways of roadways measured from the lowest or inside edges to the highest or outside edges. Superelevation rate determinations are based on:

- Design speeds
- Curve radii
- Side friction factors (found on [pages 3-53 thru 3-59, 2018 GDHS.](#))
- The method used to distribute superelevation rates (e) and side friction factors (f) - AASHTO has established five (5) alternative methods of distributing superelevation and side friction for curve radii that are larger than minimum (see [Section 3.3.2.3, Pages 3-26 thru 3-30, 2018 GDHS.](#))
- Allowed maximum superelevation rate (e_{max}).

AASHTO provides superelevation tables for design speed-radius combinations based on $e_{max} = 4\%$, 6% , 8% , 10% and 12% , which are computed based on Method 5 for distributing superelevation and side friction factors (see [Table 3-8 thru Table 3-12 on pages 3-42 thru 3-51, 2018 GDHS.](#)) These apply to rural highways and high-speed urban streets. [Exhibit 5.1](#) contains superelevation tables for $e_{max} = 4\%$ and $e_{max} = 6\%$, which are derived

from the AASHTO tables.

AASHTO also provides superelevation rates for low-speed urban streets computed based on Method 2 for distributing superelevation and side friction factors (see [Table 3-13 on pages 3-54 thru 3-57, 2018 GDHS](#)).

[Table 5.7](#) shows WisDOT policy on maximum superelevation rates (e_{max}). Do not use $e_{max} = 8\%$ except as shown in [Table 5.7](#) (Note: [Exhibit 5.1](#) does not contain superelevation tables for $e_{max} = 8\%$ - use [Table 3-10 on page 3-45, 2018 GDHS](#)). Superelevation rates greater than 8 percent are not recommended for highways in areas with ice and snow (See [page 5-4, 2018 GDHS](#)).

Do not use superelevation rates greater than 8 percent or less than 2 percent except as noted in [Table 5.7](#).

The definitions of the various highway types are as follows:

Rural highway - A highway with a rural cross section and having a posted speed of 50 mph or higher.

High-speed urban highway - Generally, a highway with curb and gutter and having a posted speed of 50 mph or higher.

Transition highway - Generally, a highway with a posted speed of 45 mph that is in a developing area between a rural highway (or high-speed urban highway) and a low-speed urban street.

Low-speed urban street - Generally, a street with curb & gutter (but some low-speed urban streets do not have curb and gutter) and having a posted speed of 40 mph or lower.

Table 5.7 WisDOT Policy on the Use of Superelevation Rate

| Areas of Application | | e_{max} | | |
|---------------------------|---|-----------------------|---------------------------|---------------------------|
| Highway Type | Work Type | Existing ^A | Design UPPER ^B | Design LOWER ^B |
| Interstate freeways | Modernization and bridge replacements (including approaches) | any | 6% | 6% |
| | Rehabilitation ^C | >8% | 8% | 6% |
| | Rehabilitation ^C | <=8% | 6% | 6% |
| | Perpetuation ^F | any | Existing | Existing |
| High-speed urban highways | Modernization and bridge replacements (including approaches) ^D | any | 6% | 4% |
| | Rehabilitation ^{C, D} | >6% | 6% | 4% |
| | Rehabilitation ^{C, D} | <=6% | 4% | 4% |
| | Perpetuation ^F | any | existing | existing |
| Transition highways | Modernization and bridge replacements (including approaches) | any | 4% | 4% |
| | Rehabilitation ^C | any | 4% | 2% |
| | Perpetuation ^F | any | existing | existing |
| Low-speed urban streets | Modernization and bridge replacements (including approaches) ^E | any | 4% | 4% |
| | Rehabilitation ^{C, E} | any | 4% | 2% |
| | Perpetuation ^F | any | existing | existing |

Notes for Superscript:

A. Determine existing e_{max} by inspecting superelevation information on as-built plans

B. For design consistency, use uniform design e_{max} for sections of roadways with the same highway types and work types - i.e., use either UPPER e_{max} or LOWER e_{max} .

Approved SCD's or DJs are required if design e_{max} for a project is greater than UPPER e_{max} or less than LOWER e_{max} .

Use superelevation tables in [Exhibit 5.1](#) for Rural Highways (except, use [Table 3-10, on page 3-45, 2018 GDHS](#) for $e_{\max} = 8\%$), High-Speed Urban Highways and Transition Highways.

Use [Figure 3-7, Page 3-58, 2018 GDHS](#), for Low-Speed Urban Streets (or consider using Exhibit 5.1 if practical).

- C. Perform safety evaluations to determine if modifying the curve radii or superelevation rates are needed to address crash problems which are occurring, and that the improvements are warranted based on acceptable benefit/costs per [FDM 11-38](#).
- D. Superelevation rates on high-speed urban roadways are preferably based on $e_{\max} = 4\%$. Consider adverse effects caused by factors such as, cross street profile site conditions that include driveways, sidewalks, or other intersections. An $e_{\max} = 6\%$ may be used if minimal adverse effects are caused as a result and there are no traffic stops or signals present or anticipated in the future.
- E. Superelevation rates for low-speed urban streets should not exceed 4 percent. At lower non-uniform running speeds, which are typical in urban areas, drivers are more tolerant of discomfort, thus permitting employment of increased amounts of side friction ([Method 2, pages 3-26 thru 3-30, 2018 GDHS](#)) for use in design of horizontal curves.
[Note: The results from the nomograph on [page 3-58, 2018 GDHS](#) differ considerably from the nomograph in the 2001 GDHS²⁷ (Exhibit 3-40 on p. 196). Low-speed urban streets with existing superelevation that meets the requirements of the 2001 GDHS may retain those superelevations that are impractical to upgrade to the superelevations obtained from the nomograph on [page 3-58, 2018 GDHS](#), unless there is an unacceptable history of curve related crashes. Document this in the Design Study Report.]
- F. Superelevation rates for Perpetuation projects with crash histories in which it has been determined that safety mitigation measures are appropriate, should first determine if the roadway pavement cross sections can be milled or wedged to improve the superelevations within the outside edges of the shoulder points, and if not, if other safety mitigation measures, such as surface friction treatments, shoulder pavement widening, or signing and marking, etc., are appropriate.

Consider modifying cross street approach grade(s) at signalized intersections if superelevations on the mainlines result in unacceptable break-over angles between the mainline edges of pavements and the cross-street approach profiles.

Very flat horizontal curves on rural or high-speed urban highways require no superelevations. Traffic entering curves to the right have some superelevations in the normal crown slopes. Traffic entering curves to the left have adverse or negative superelevations. Lack of adequate superelevations, where needed, can result in undesirable conditions including loss of safety factor between the side frictions available versus side frictions used and/or driver failure to maintain appropriate lateral position within the lanes and increased occupant discomfort. The minimum curve radii for "Normal Crown" which can be designed without superelevation for open road conditions are shown in [Exhibit 5.1](#) for various design speeds.

5.3.3 Superelevation Transitions

Superelevation transition is the length required to rotate the cross slope of a highway from a normal crowned slope to a fully super-elevated cross slope. See the illustration on [Attachment 5.8](#) and [Attachment 5.9](#). This transition includes a "tangent runoff" length needed to remove or add adverse crown. The rotation of the planes of highways to achieve super-elevated roadways through horizontal curves begins on the tangent approaches to the curves. WisDOT practice is to place the tangent runouts and approximately two-thirds of the lengths of runoffs on the tangent approaches and one-third of the lengths of runoffs on the curves. For undivided highways, the axis of rotation is the centerlines of the pavements (see [Attachment 5.8](#)). On divided highways, the axis of rotation is normally the median edges of the pavements (see [Attachment 5.9](#)). Provide Vertical curves of sufficient lengths to ensure smooth pavement edges and centerline profiles within the superelevation transitions ([Section 3.3.8.7 of the 2018 GDHS](#) suggests minimum lengths in feet equal to the design speeds in mph). These curves may be either computed or determined graphically.

When using the superelevation rates from the tables in [Exhibit 5.1](#), use the corresponding values for "L" and "T" from these tables to design the curves. Small increases in runoffs may be appropriate on high-type facilities (freeways, expressways, or other divided highways) in order to facilitate drainage or to smooth out the traveled way edge profiles. Any superelevation transition locations that are computed using something other than the superelevation tables and runoff tables provided, must be hand entered into the superelevation spreadsheet for

²⁷ A Policy on Geometric Design of Highways and Streets 2001, (2nd printing) 4th edition. AASHTO, 2001.

consideration by Civil-3D. For the above example, the designer was assumed to choose a superelevation rate of 2 percent. Compute the theoretical point of normal crown and the theoretical point of full superelevation.

Given:

PC = Station 870+00.00

$L = (wn_1)e_d(b_w) / \Delta$, where

w = lane width (feet) = 12-feet (use for consistency and practicality even if lane width used does not equal 12-feet);

n_1 = number of lanes rotated = 1;

e_d = design superelevation rate (%) = 2.0%;

b_w = adjustment factor for number of lanes rotated (see Table on page 9 of Exhibit 5.1) = 1.0;

Δ = maximum relative gradient (%) (see [Section 3.3.8.2.1 in 2018 GDHS](#)) = $(1/175) * 100 = 0.57\%$ for 40 mph

therefore, $L = 12 * 1 * 2.0\% * 1.0 / .057\% = 41.4$ ft (round to 41 ft)

$X = e_{NC}L / e_d = 41 * 0.02 / 0.02 = 41$ ft

Theoretical point of normal crown (see Figure 7 and 8):

PC - $2/3L$ - X = 870+00.00 - 27.33 - 41 =

Station 869+31.67

Theoretical point of full superelevation (see Figure 7 and 8):

PC + $1/3L$ = 870+00.00 + 13.67 =

Station 870+13.67

Where:

PC = Point of Curvature

L = Length of Runoff

X = Length of Tangent Runout

e_{NC} = Normal Crown of 2%

e_d = superelevation rate

Avoid superelevation transitions on bridges because they complicate bridge designs and construction. When superelevation must be partially developed on bridges, there should be clear understanding between the bridge designers and the roadway designers as to the method used for transition development and the resulting grades.

Auxiliary lane pavements (or right turn lanes) on the high sides of super-elevated curves should be maintained at the same slopes as the adjacent traffic lanes until the superelevation reaches 4 percent. When superelevation on the traffic lane pavements are greater than 4 percent the auxiliary lane slopes will remain constant at 4 percent. In some isolated situations where the increasing elevations of the intersecting side roads approaches the main line roads, it may be desirable to flatten the super-elevated auxiliary lanes to form gradual transitions between the super-elevated sections and the side roads. Do not exceed rollover rates greater than 5 percent between adjacent travel lanes or auxiliary lanes.

On super-elevated divided highways where "narrow" medians are present, it may be desirable to rollover the high side to reduce the elevation differences between the divided highways. (See [2018 GDHS Figure 4-2, Case A, B, or C](#)). These special situations may be desirable in urban conditions when the highways are divided by barrier walls.

Do not exceed rollover rates greater than 8 percent between shoulders and travel lanes or auxiliary lanes.

5.4 Vertical Alignments

Highway vertical alignments consists of tangents or grades and vertical curves. Vertical curves are based on sight distance considerations. Headlight sight distances are the primary factors used to determine the lengths of sag vertical curves (see [Attachment 5.5](#)).

Although grade changes without vertical curves are discouraged, there may be situations where they are necessary. These must be explained and justified in the SCDs or DSRs. [Table 5.8](#) shows the maximum changes in grades without vertical curves. Some rounding of the deflection points is anticipated during construction.

Table 5.8 Maximum Changes in Grade Without Vertical Curves

| Design Speed mph | 20 | 30 | 40 | 45 | 50 | 60 | 65 | 70 |
|---------------------------------------|------|------|------|------|------|------|------|------|
| Maximum Change in Grade in Percent | 1.20 | 1.00 | 0.80 | 0.70 | 0.60 | 0.40 | 0.30 | 0.20 |

5.4.1 Grades

Maximum grades (see [Attachment 5.3](#) of this procedure and [FDM 11-15 Attachment 1.4](#)) vary with terrains, design speeds and functional classifications.

The minimum grades on roadways with rural cross sections is 0.0 percent, i.e., flat, except in areas of super-elevation transitions and other areas with pavement rotations. Do not use flat grades in areas of superelevation transitions and other areas with pavement rotations because the combinations of flat longitudinal grades with flat cross-slopes may result in pavement surface drainage problems. Provide minimum grades in these areas based on GDHS guidance for “Minimum Transition Grades”²⁸. This applies to both rural and urban roadways.

If grades of less than 0.5 percent are used, then side ditches should be specially designed to provide sufficient longitudinal gradients for drainage. On divided highways, grade lines of opposing roadways should be treated independently except where topographic or other conditions require them to be identical. The minimum gradients on structures is 0.5 percent to ensure positive drainage.

Compatibility of curb and gutter grades with existing developments is essential in reducing damage to abutting properties and the amount of right-of-way to be acquired. To ensure drainage the minimum gradients of curb and gutters are desirably 0.50 percent but at least 0.30 percent. Special attention may be required to assure proper drainage of curbed pavements at the apex of crest vertical curves where level points occur. Drainage should be adequate for vertical curves having “k” values of 167 or less (see [Attachment 5.4](#) and [Page 3-175, 2018 GDHS](#)).

SCD or DJ documentation is required for grades that are either greater than maximum or less than minimum.

5.4.1.1 Climbing Lanes

See [FDM 11-15-10](#) for guidance on climbing lanes.

5.4.2 Vertical Curves

Design vertical curves to provide adequate sight distances, safety, comfortable driving, good drainage, and pleasing appearances. They are normally symmetrical parabolas. Notable exceptions would be the use of asymmetrical parabolic curves to provide better drainage of structures located on crest vertical curves.

Vertical curves are generally identified by their “K” values. K is the rate of curvature and is defined as the length of the vertical curve (L) divided by the algebraic difference in grade (A); i.e. the horizontal distance in feet required for a 1 percent change in gradient. K is affected by sight distances, comfort, drainage, and aesthetic quality. Sight distances and vertical curve k-values are shown in [Attachment 5.4](#) through [Attachment 5.5](#) for each of the sight distance categories discussed earlier in [FDM 11-10-5.1.1.1](#) “Application of Stopping Sight Distances (SSDs) and Decision Sight Distances (DSDs)”. Crest vertical curve values are shown in [Attachment 5.4](#). Sag vertical curve values are shown in [Attachment 5.5](#).

Compute sight distances (S) on vertical curves by re-arranging the equations on [Attachment 5.4](#) and [Attachment 5.6](#) to solve for (S). Note that as vehicles traverse vertical curves, the distances to the ends of the vertical curves (i.e., the L-dimension in the equation) get shorter. The A-dimensions also decrease because $A=L/K$ and K is constant. The sight distances on the vertical curves begin to increase when the vehicles reach points where $S>L$. Consider this when determining if the requirements for sight distance categories are met.

SCD or DJ documentation for stopping sight distances are required if crest vertical curves do not provide minimum sight distances for the sight distance categories.

²⁸ See *A Policy on Geometric Design of Highways and Streets*, 7th edition, AASHTO, 2018, [Page 3-87](#).

SCD or DJ documentation is required for sag vertical curves if:

- They do not provide the minimum headlight sight distances and adequate street lighting is not provided, or
- They do not meet the comfort criteria for sag vertical curves

On vertical curves with $K > 167$, there will be sections of roadways at least 100 feet in length near the crests or sags with grades of less than 0.30%. These conditions may create drainage problems, especially on curbed highways. It is not intended that K of 167 feet per percent grade be considered a design maximum, but merely a value beyond which drainage should be more carefully designed.

5.4.3 Vertical Clearances

See [FDM 11-35-1.5](#), [FDM 11-35 Attachment 1.8](#) and [FDM 11-35 Attachment 1.9](#) for vertical clearance design criteria for different combinations of overpass and underpass facilities.

Vertical clearances for overhead utility facilities shall comply with all applicable state and national electrical codes. See WisDOT Highway Maintenance Manual chapter 9-15-25 section 2.2 ([HMM 9-15-25](#)) and WisDOT [Bridge Manual Chapter 3](#) requirements for vertical clearance for overhead utilities.

The Department has adopted OSOW High Clearance Routes with the objective of minimizing overhead constraints for OSOW vehicles along these routes (including to sign structures, traffic signal monotube arms, overhead utilities, and other overhead appurtenances). Minimum 20'-0" vertical clearances are needed along these high clearance routes for these overhead constraints. In addition, the Department's goal is to provide minimum vertical clearances of 20'-0" for bridges and at railroad crossings along these routes. See the OSOW maps for routes designated as High Clearance routes.

<https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrcs/tools/planning-maps.aspx>

See the following for additional guidance and requirements for OSOW High Clearance Routes:

- [FDM 11-20-1.9](#) - Clearances for Urban Roadways
- [FDM 11-20-1.9.5](#) - Traffic Signal Supports (urban)
- [FDM 11-20-1.9.6](#) - Railroad Warning Signs and Signals (urban)
- [FDM 11-25-1.4.1](#) - OSOW High Clearance Routes
- [FDM 11-25-40.1](#) - Railroad Crossings (coordination)
- [FDM 11-35-1.5.1](#) - OSOW High Clearance Routes
- [FDM 11-35 Attachment 1.8](#) - Minimum Vertical Clearance for New Bridges and Replacement Bridges
- [FDM 11-35 Attachment 1.9](#) - Minimum Vertical Clearance for Bridges to Remain

LIST OF ATTACHMENTS

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| Attachment 5.2 | Sight Distance Category Applications |
| Attachment 5.3 | Maximum Grades by Functional Classification |
| Attachment 5.4 | Sight Distance for Crest Vertical Curves |
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| Attachment 5.8 | Superelevation Transition of Two-Lane Highway to the Right |
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LIST OF EXHIBITS

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| Exhibit 5.1 | Superelevation Tables (emax = 4% and 6%) |
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