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Chapter 3

Geometric Design

Figures

Tables

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3 Geometric Design

3.1 Introduction

Geometric design is defined as the design or proportioning of the visible elements of the street or highway. The geometry of the street or highway is of central importance since it provides the framework for the design of other highway elements. In addition, the geometric design establishes the basic nature and quality of the vehicle path, which has a primary effect upon the overall safety characteristics of the street or highway.

The design of roadway geometry must be conducted in close coordination with other design elements of the street or highway. These other elements include pavement design, roadway lighting, traffic control devices, transit, drainage, and structural design. The design should consider safe roadside clear zones, pedestrian safety, emergency response, and maintenance capabilities.

The safety characteristics of the design should be given primary consideration. The initial establishment of sufficient right of way and adequate horizontal and vertical alignment is not only essential from a safety standpoint, but also necessary to allow future upgrading and expansion without exorbitant expenditure of highway funds.

The design elements selected should be reasonably uniform but should not be inflexible.

The minimum standards presented in this chapter should not automatically become the standards for geometric design. The designer should consider use of a higher level, when practical, and consider cost-benefits as well as consistency with adjacent facilities. Reconstruction and maintenance of facilities should, where practical, include upgrading to these minimum standards.

In restricted or unusual conditions, it may not be possible to meet the minimum standards. In such cases, the designer shall obtain an exception in accordance with ***Chapter 14 – Design Exceptions and Variations*** from the reviewing or permitting organization. However, every effort should be made to obtain the best possible alignment, grade, sight distance, and proper drainage consistent with the terrain, the development, safety, and fund availability. The concept of road users has expanded in recent years creating additional considerations for the designer.

In making decisions on the standards to be applied to a particular project, the designer must also address the needs of pedestrians, bicyclists, elder road and transit users, people with disabilities, freight movement and other users and uses. This is true for both urban and rural facilities.

The design features of urban local streets are governed by practical limitations to a greater extent than those of similar roads in rural areas. The two dominant design controls are: (1) the type and extent of urban development and its limitations on rights of way and (2) zoning or regulatory restrictions. Some streets primarily are land service streets in residential areas. In such cases, the overriding consideration is to foster a safe and pleasing environment. Other streets are land service only in part, and features of traffic and public transit service may be predominant.

The selection of the type and exact design details of a particular street or highway requires considerable study and thought. When specific criteria are not provided in this Manual and reference is made to guidelines and design details given by current American Association of State Highway and Transportation Officials (AASHTO) publications, these guidelines and standards should generally be considered as minimum criteria. For the design of recreational roads, local service roads, and alleys, see ~~***A Policy on Geometric Design of Highways and Streets (AASHTO, 2011)***~~***A Policy on Geometric Design of Highways and Streets (AASHTO, 2018)***, also known as the ~~***AASHTO Greenbook (2011)***~~***AASHTO Green Book (2018)*** and other publications.

Right of way and pavement width requirements for new construction may be reduced for the paving of certain existing unpaved streets and very low volume rural roads provided all the conditions listed below are satisfied:

- The road is functionally classified as a local road.
- The 20-year projected ADT is less than or equal to 400 vehicles per day and the design year projected peak hourly volume is 100 vehicles per hour or less. Note: The design year may be any time within a range of the present to 20 years in the future, depending on the nature of the improvement.
- The road has no foreseeable probability of changing to a higher functional classification through changes in land use, extensions to serve new developing land areas, or any other use which would generate daily or hourly traffic volumes greater than those listed above.
- There is no reasonable possibility of acquiring additional right of way without.
- Incurring expenditures of public funds in an amount which would be excessive compared to the public benefits achieved.
- Causing substantial damage or disruption to abutting property improvements to a degree that is unacceptable considering the local environment

3.2 Objectives

The major objective in geometric design is to establish a vehicle path and environment providing a reasonable margin of safety for the motorist, transit, bicyclist, and pedestrian under the expected operating conditions and speed. It is recognized that Florida's design driver is aging, and tourism is our major industry. This gives even more emphasis on simplicity and easily understood geometry. The design of street or highway features should consider the following:

- Provide the most simple geometry attainable, consistent with the physical constraints
- Provide a design that has a reasonable and consistent margin of safety at the expected operating speed
- Provide a design that is safe at night and under adverse weather conditions
- Provide a facility that is adequate for the expected traffic conditions and transit needs
- Allow for reasonable deficiencies in the driver, such as:
    - Periodic inattention
    - Reduced skill and judgment
    - Slow reaction and response
- Provide an environment that minimizes hazards, is as hazard free as practical, and is "forgiving" to a vehicle that has deviated from the travel path or is out of control.

3.3 Design Elements

3.3.1 Design Speed

Design speed is a selected speed used to determine the various geometric design features of the street or highway. Selection of an appropriate design speed must consider the anticipated operating speed, topography, existing and future adjacent land use, and functional classification. Consideration must also be given to pedestrian and bicycle usage.

Many critical design features such as sight distance and curvature are directly related to, and vary appreciably with, design speed. For this reason, the selected design speed should be consistent with the speeds that drivers are likely to expect on a given street or highway facility. The design speed shall not be less than the expected posted or legal speed limit. Once the design speed is selected, all pertinent highway features should be related to it to obtain a balanced design.

Above minimum design criteria for specific design elements such as flatter curves and longer sight distances should be used where practical, particularly on high speed facilities. On lower speed facilities, use of above minimum values may encourage travel at speeds higher than the design speed.

The design speed utilized should be consistent over a given section of street or highway. Required changes in design speed should be effected in a gradual fashion. When isolated reductions in design speed cannot reasonably be avoided, appropriate speed signs should be posted.

Minimum and maximum values for design speed are given in **Table 3 – 1 Minimum and Maximum Design Speed**.

High speed facilities are defined as those facilities with design speeds 50 mph and greater. Low speed facilities are defined as those facilities with design speeds 45 mph and less. The posted speed shall be less than or equal to the design speed.

The ***AASHTO Green Book (2018)****~~AASHTO Greenbook (2011)~~* provides additional information on design speed.

Table 3-1 Minimum and Maximum Design Speed (mph)

Facility[1]		AADT (vpd)	Terrain	Design Speed (mph)
Freeways	Rural	All	Level and Rolling	70
	Urban	All	Level and Rolling	50 – 70[2]
Arterials	Rural	All	Level	60 – 70
			Rolling	50 – 70
	Urban	All	All	30 – 60[3]
Collectors	Rural	≥ 400	Level	60 – 65 (50 mph min for AADT 400 to 2000)
			Rolling	50 – 65 (40 mph min for AADT 400 to 2000)
		< 400	Level	40 – 60
			Rolling	30 – 60
	Urban	All	All	30 – 50[3]
Local	Rural	≥ 400	Level	50 – 60
			Rolling	40 – 60
		< 400	Level	30 – 50
			Rolling	20 – 40
	Urban	All	All	20 – 30[4]

Notes:

1. Urban design speeds are applicable to streets and highways located within designated urban boundaries as well as those streets and highways outside designated urban boundaries yet within small communities or urban like developed areas. Rural design speeds are applicable to all other rural areas.
2. A design speed of 70 mph should be used for urban freeways when practical. Lower design speeds should only be used in highly developed areas with closely spaced interchanges. For these areas a minimum design speed of 60 mph is recommended unless it can be shown lower speeds will be consistent with driver expectancy.
3. Lower speeds apply to central business districts and in more developed areas while higher speeds are more applicable to outlying and developing areas.
4. Since the function of urban local streets is to provide access to adjacent property, all design elements should be consistent with the character of activity on and adjacent to the street, and should encourage speeds generally not exceeding 30 mph.

3.3.2 Design Vehicles

A "design vehicle" is a vehicle with representative weight, dimensions, and operating characteristics, used to establish street and highway design controls for accommodating vehicles of designated classes. For the purpose of geometric design, the design vehicle should be one with dimensions and minimum turning radii larger than those of almost all vehicles in its class. Design vehicles are listed in **Table 3 – 2 Design Vehicles**. One or more of these vehicles should be used as a control in the selection of geometric design elements. In certain industrial (or other) areas, special service vehicles may have to be considered in the design. Fire equipment and emergency vehicles should have reasonable access to all areas. Additional information on the maximum width, height and length of vehicles in Florida can be found in ***Section 316.515, F.S. Motor Vehicles; Maximum width, height, length***.

If a significant number or percentage (5 percent of all the total traffic) of vehicles of those classes larger than passenger vehicles are likely to use a particular street or highway, that class should be used as a design control. The design of arterial streets and highways should normally be adequate to accommodate all design vehicles. The decision as to which of the design vehicles (or other special vehicles) should be used as a control is complex and requires careful study. Each situation must be evaluated individually to arrive at a reasonable estimate of the type and volume of expected traffic.

- Design criteria significantly affected by the type of vehicle include:
- Horizontal and vertical clearances
- Alignment
- Lane widening on curves
- Shoulder width requirements
- Turning roadway and intersection radii
- Intersection sight distance
- Acceleration criteria

Particular care should be taken in establishing the radii at intersections, so vehicles may enter the street or highway without encroaching on adjacent travel lanes or leaving the pavement. It is acceptable for occasional trucks or buses to make use of both receiving lanes, especially on side streets.

Table 3-2 Design Vehicles

Design Vehicle		Dimensions (feet)					
Type	**Symbol**	**Wheelbase**	**Overhang**		**Overall Length**	**Overall Width**	**Height**
			Front	**Rear**			
Passenger Car	P	11	3	5	19	7	4.3
Single Unit Truck	SU-30	20	4	6	30	8	11-13.5
Single Unit Truck – 3 Axle	SU-40	25	4	10.5	39.5	8	11-13.5
City Transit Bus	CITY-BUS	25	7	8	40	8.5	10.5
Conventional School Bus (65 passenger)	S-BUS 36	21.3	2.5	12.0	35.8	8.0	10.5
Articulated Bus	A-BUS	22+19.4=41.4	8.6	10	60	8.5	11
Motor Home	MH	20	4	6	30	8	12
Car & Camper Trailer	P/T	11+5+17.7=33.7**	3	12	48.7	8	10
Car & Boat Trailer	P/B	11+5+15=31**	3	8	42	8	---
Intermediate Semitrailer	WB-40	12.5+25.5=38	3	4.5	45.5	8	13.5
Intermediate Semitrailer	WB-50	14.6+35.4=50	3	2	55	8.5	13.5
Interstate Semitrailer***	WB-62	19.5+41=60.5	4	4.5	69	8.5	13.5
Florida Interstate Semitrailer***	WB-62FL	19.5+41=60.5	4	9	73.5	8.5	13.5
Interstate Semitrailer***	WB-67	~~21.6~~19.5+45.~~4~~5= ~~67~~65	4	~~2~~4.5	73.5	8.5	13.5
"Double-Bottom"-Semitrailer/ Trailer Combination	WB-67D	11+23+10*+22.5= 66.5	2.3	3.0	72.3	8.5	13.5

Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Design Controls and Criteria, Table 2-4~~1~~a~~b~~.

* Distance between rear wheels of front trailer and front wheels of rear trailer

** Distance between rear wheels of trailer and front wheels of car

*** The term "Interstate" does not imply the vehicle is restricted to interstate and limited access highways only.

The minimum turning radii of design vehicles is presented in **Table 3 – 3 Minimum Turning Radii of Design Vehicles**. The principal dimensions affecting design are the minimum centerline turning radius, the out-to-out track width, the wheelbase, and the path of the inner rear tire. The speed of the turning vehicle is assumed to be less than 10 mph.

The boundaries of the turning path of each design vehicle for its sharpest turns are established by the outer trace of the front overhang and path of the inner rear wheel. This sharpest turn assumes that the outer front wheel follows the circular arc defining the minimum centerline turning radius as determined by the vehicle steering mechanism.

Figures illustrating the minimum turning radii for a variety of vehicles along with additional information can be found in the ~~***AASHTO Greenbook (2011), Chapter 2 – Design Controls and Geometrics***~~***AASHTO Greenbook (2018), Chapter 2 – Design Controls and Geometrics***.

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Table 3-3 Minimum Turning Radii of Design Vehicles

Design Vehicle		Dimensions In Feet		
Type	**Symbol**	**Minimum Design Turning Radius**	**Centerline Turning* Radius**	**Minimum Inside Radius**
Passenger Car	P	23.8	21.0	14.4
Single Unit Truck	SU-30	41.8	38.0	28.4
Single Unit Truck – 3 Axle	SU-40	51.2	47.4	36.4
City Transit Bus	CITY-BUS	41.6	37.8	24.5
Conventional School Bus (65 passenger)	S-BUS 36	38.6	34.9	23.8
Articulated Bus	A-BUS	39.4	35.5	21.3
Motor Home	MH	39.7	36.0	26.0
Car & Camper Trailer	P/T	32.9	30.0	18.3
Car & Boat Trailer	P/B	23.8	21.0	8.0
Intermediate Semitrailer	WB-40	39.9	36.0	19.3
Intermediate Semitrailer	WB-50	45	41	17.0
Interstate Semitrailer	WB-62	44.8	41.0	7.4
Florida Interstate Semitrailer***	WB-62FL	44.8	41.0	7.4
"Double-Bottom"-Semitrailer/Trailer Combination	WB-67D	44.8	40.9	19.1

Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Design Controls and Criteria, Table 2-5a~~2b~~.

* The turning radius assumed by a designer when investigating possible turning paths and is set at the centerline of the front axle of a vehicle. If the minimum turning path is assumed, the CTR approximately equals the minimum design turning radius minus one-half the front width of the vehicle.

3.3.3 Sight Distance

The provision for adequate horizontal and vertical sight distance is an essential factor in the development of a safe street or highway. An unobstructed view of the upcoming roadway is necessary to allow time and space for the safe execution of passing, stopping, intersection movements, and other normal and emergency maneuvers. It is also important to provide as great a sight distance as possible to allow the driver time to plan for future actions. The driver is continuously required to execute normal slowing, turning, and acceleration maneuvers. If he can plan in advance for these actions, traffic flow will be smoother and less hazardous. Unexpected emergency maneuvers will also be less hazardous if they are not combined with uncertainty regarding the required normal maneuvers. The appropriate use of lighting ***(Chapter 6 – Lighting)*** may be required to provide adequate sight distances for night driving.

Future obstruction to sight distance that may develop (e.g., vegetation) or be constructed should be taken into consideration in the initial design. Areas outside of the road right of way that are not under the highway agency's jurisdiction should be considered as points of obstruction. Planned future construction of median barriers, guardrails, grade separations, or other structures should also be considered as possible sight obstructions.

3.3.3.1 Stopping Sight Distance

Safe stopping sight distances shall be provided continuously on all streets and highways. The factors, which determine the minimum distance required to stop, include:

- Vehicle speed
- Driver's total reaction time
- Characteristics and conditions of the vehicle
- Friction capabilities between the tires and the roadway surface
- Vertical and horizontal alignment of the roadway

It is desirable that the driver be given sufficient sight distance to avoid an object or slow-moving vehicle with a natural, smooth maneuver rather than an extreme or panic reaction.

The determination of available stopping sight distance shall be based on a height of the driver's eye equal to 3.50 feet and a height of obstruction to be avoided equal to 2.0 feet. It would, of course, be desirable to use a height of obstruction equal to zero (coincident with the roadway surface) to provide the driver with a more positive sight condition. Where horizontal sight distance may be obstructed on curves, the driver's eye and the obstruction shall be assumed to be located at the centerline of the traffic lane on the inside of the curve.

The stopping sight distance shall be no less than the values given in **Table 3 – 4 Minimum Stopping Sight Distance** for level and rolling roadways.

Table 3-4 Minimum Stopping Sight Distance

Design Speed (mph)	Stopping Sight Distance (feet)						
	Level (≤ 2%)	Downgrades			Upgrades		
		3%	6%	9%	3%	6%	9%
15	80	80	82	85	75	74	73
20	115	116	120	126	109	107	104
25	155	158	165	173	147	143	140
30	200	205	215	227	200	184	179
35	250	257	271	287	237	229	222
40	305	315	333	354	289	278	269
45	360	378	400	427	344	331	320
50	425	446	474	507	405	388	375
55	495	520	553	593	469	450	433
60	570	598	638	686	538	515	495
65	645	682	728	785	612	584	561
70	730	771	825	891	690	658	631

Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Table 3-1 Stopping Sight Distance on Level Roadways and Table 3-2 Stopping Sight Distance on Grades.

3.3.3.2 Decision Sight Distance

Decision sight distance is the distance needed for a driver to detect an unexpected or otherwise difficult to perceive information source or condition in a roadway environment that may be visually cluttered. It allows the driver to recognize the condition or its potential threat, select an appropriate speed and path, and initiate and complete complex maneuvers. Minimum stopping distance does not provide sufficient space or time for the driver to make decisions regarding complex situations requiring more than simple perception-reaction process.

Examples of critical locations where additional sight distance is needed include interchange and intersection locations, where unusual or unexpected maneuvers are needed, changes in typical sections such as toll plazas or lane drops, and areas of concentrated demand where there is visual noise from competing sources of information, such as roadway elements, traffic, traffic control devices and advertising signs.

The decision sight distances in **Table 3 – 5 Decision Sight Distance** may be used (1) to provide values for sight distances that may be appropriate at critical locations, and (2) to serve as criteria for evaluating the suitability of the available sight distances at these locations. If it is not practical to provide decision sight distance because of horizontal or vertical curvature or if relocation of decision points is not practical, special attention should be given to using appropriate traffic control devices providing advance warning of the conditions that are likely to be encountered.

Table 3-5 Decision Sight Distance

Design Speed (mph)	Decision Sight Distance (feet)				
	Level Avoidance Maneuver				
	A	B	C	D	E
20	130	305	300	355	410
25	170	395	375	445	515
30	220	490	450	535	620
35	275	590	525	625	720
40	330	690	600	715	825
45	395	800	675	800	930
50	465	910	750	890	1030
55	535	1030	865	980	1135
60	610	1150	990	1125	1280
65	695	1275	1050	1220	1365
70	780	1410	1105	1275	1445

Source: AASHTO Green Book (2018)~~2011 AASHTO Greenbook~~, Table 3 - 3 Decision Sight Distance

Notes:
1. Avoidance Maneuver A: Stop on rural road – t = 3.0 s
2. Avoidance Maneuver B: Stop on urban road – t = 9.1 s
3. Avoidance Maneuver C: Speed/path/direction change on rural road – t varies between 10.2 and 11.2 s
4. Avoidance Maneuver D: Speed/path/direction change on suburban road – t varies between 12.1 and 12.9 s
5. Avoidance Maneuver E: Speed/path/direction change on urban road – t varies between 14.0 and 14.5 s

The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum by 25 percent or more. A minimum sight distance of 1000 feet, measured from the driver's eye to the road surface is a desirable goal. There should be a clear view of the exit terminal including the exit nose.

3.3.3.3 Passing Sight Distance

The passing maneuver, which requires occupation of the opposing travel lane, is inherently dangerous. The driver is required to make simultaneous estimates of time, distance, relative speeds, and vehicle capabilities. Errors in these estimates result in frequent and serious crashes.

Streets or highways with two or more travel lanes in a given direction are not subject to requirements for safe passing sight distance. Two-lane, two-way highways should be provided with safe passing sight distance for as much of the highway as feasible. The driver demand for passing opportunity is high and serious limitations on the opportunity for passing reduces the capacity and safe characteristics of the highway.

The distance traveled after the driver's final decision to pass (while encroaching into the opposite travel path) is that which is required to pass and return to the original travel lane in front of the overtaken vehicle. In addition to this distance, the safe passing sight distance must include the distance traveled by an opposing vehicle during this time period, as well as a reasonable margin of safety. Due to the many variables in vehicle characteristics and driver behavior, the passing sight distance should be as long as is practicable.

The determination of passing sight distance shall be based on a height of eye equal to 3.50 feet and a height of object passing equal to 3.50 feet. Where passing is permitted, the passing sight distance shall be no less than the values given in **Table 3 – 6 Minimum Passing Sight Distance**.

Table 3-6 Minimum Passing Sight Distance

Design Speed (mph)	20	25	30	35	40	45	50	55	60	65	70
Minimum Passing Sight Distance (feet)	400	450	500	550	600	700	800	900	1000	1100	1200

Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Table 3-4 Passing Sight Distance for Design of Two-Lane Highways.

Note:
For application of passing sight distance, use an eye height of 3.50 feet and an object height of 3.50 feet above the road surface.

3.3.3.4 Intersection Sight Distance

Sight distances for intersection movements are given in the general intersection requirements (C.9 Intersection Design, this chapter).

3.3.4 Horizontal Alignment

3.3.4.1 General Criteria

The standard of alignment selected for a particular section of street or highway should extend throughout the section with no sudden changes from easy to sharp curvature. Where sharper curvature is unavoidable, a sequence of curves of increasing degree should be utilized.

Winding alignment consisting of sharp curves is hazardous, reduces capacity, and should be avoided. The use of as flat a curve as possible is recommended. Flatter curves are not only less hazardous, but also frequently less costly due to the shortened roadway.

Maximum curvature should not be used in the following locations:

- High fills or elevated structures. The lack of surrounding objects reduces the driver's perception of the roadway alignment.
- At or near a crest in grade.
- At or near a low point in a sag or grade.
- At the end of long tangents.
- At or near intersections, transit stops, or points of ingress or egress.
- At or near other decision points.

The "broken back" arrangement of curves (short tangent between two curves in the same direction) should be avoided. This is acceptable only at design speeds of 30 mph or less. This arrangement produces unexpected and hazardous situations.

When reversals in alignment are used and superelevation is required, a sufficient length of tangent between the reverse curves is required for adequate superelevation transition.

Compound curves should be avoided, especially when curves are sharp. They tend to produce erratic and dangerous vehicle operations. When compound curves are necessary, the radius of the flatter curve should not be more than 50 percent greater than the sharper curve.

The transition between tangents and curves should normally be accomplished by the use of appropriate straight-line transitions or spirals. This is essential to assist the driver in maintaining his vehicle in the proper travel path.

3.3.4.2 Maximum Deflections in Alignment without Curves

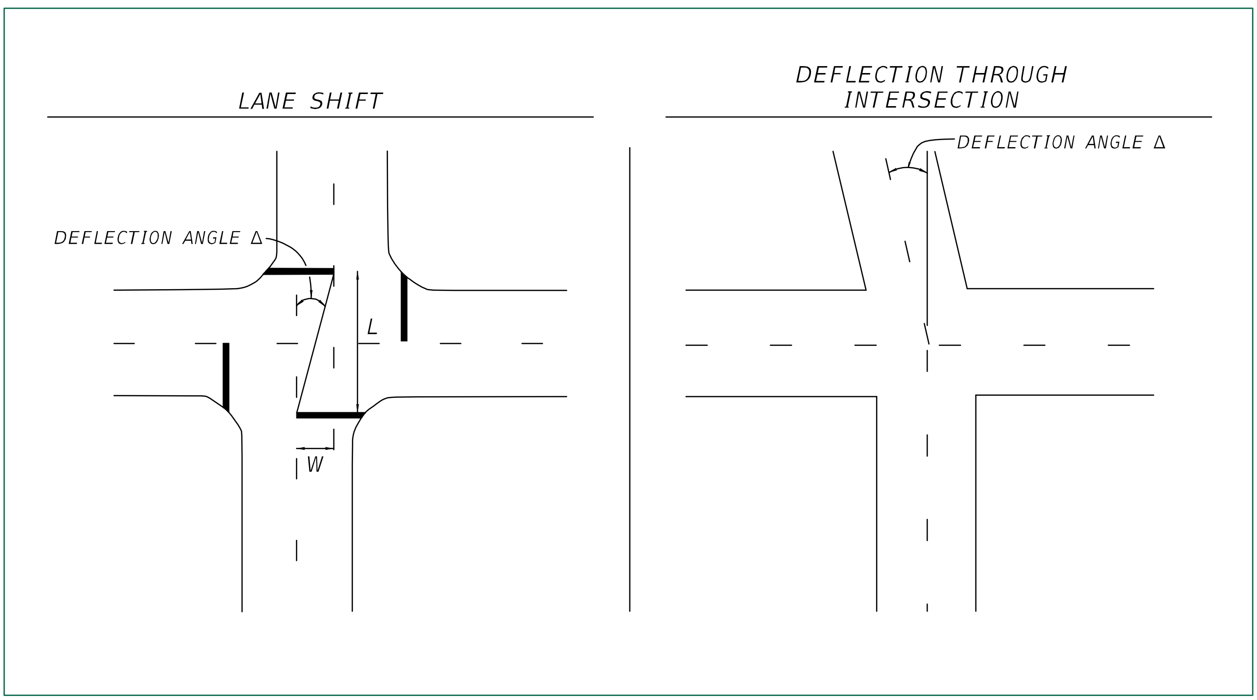
The point where tangents intersect is known as the point of intersection (PI). Although the use of a PI with no horizontal curve is discouraged, there may be conditions where it is necessary. The maximum deflection criteria without a horizontal curve are as follows:

- Flush shoulder and curbed roadways with design speed 40 mph and less is 2° 00' 00".
- Flush shoulder roadways with design speed 45 mph and greater is 0° 45' 00".
- Curbed roadways with design speed 45 mph and greater is 1° 00' 00".
- High speed curbed roadways with design speed 50 mph and greater is 0° 45' 00".

Although deflections through intersections are discouraged, there may be conditions where it is necessary. The maximum deflection angles at intersections to be used in establishing the horizontal alignment are given in **Table 3 – 7 Maximum Deflection Angle Through Intersection**.

Table 3-7 Maximum Deflection Angle Through Intersection

Design Speed (mph)					
≤ 20	**25**	**30**	**35**	**40**	**45**
16° 00'	11° 00'	8° 00'	6° 00'	5° 00'	3° 00'



Notes:

1. The deflection angle used is not to cause a lane shift (W) of more than 6 feet from stop bar to stop bar.

Curves on main roadways should be sufficiently long to avoid the appearance of a kink. Gently flowing alignment is generally more pleasing in appearance, as well as, superior from a safety standpoint. Flatter curvature with shorter tangents is preferable to sharp curves connected by long tangents, i.e., avoid using minimum horizontal curve lengths. **Table 3-8 Minimum Lengths of Horizontal Curves** provides minimum horizontal curve lengths that should be used in establishing the horizontal alignment.

Table 3-8 Minimum Lengths of Horizontal Curves

Curve Length Based on Design Speed										
Design Speed (mph)	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**
Arterials, Collectors (Length in feet = 15 x Design Speed, but not less than 400 feet)	400	450	525	600	675	750	825	900	975	1050
Freeways - Mainline (Length in feet = 30 x Design Speed)	--	--	--	--	--	1500	1650	1800	1950	2100

Curve Length Based on Deflection Angle					
Deflection Angle (degrees)	5°	4°	3°	2°	1°
Curve Length (feet)	500	600	700	800	900

Notes:

1. Horizontal curve length should be the greater of the lengths based on design speed and length based on deflection angle.
2. If the curve lengths for arterials and collectors cannot be attained, provide the greatest attainable length possible, but not less than 400 feet.
3. If the curve lengths for mainline freeways cannot be attained, provide the greatest attainable length possible, but not less than the lengths used for arterials and collectors.
4. Curve length shall provide for full superelevation within the curve of not less than 200 feet (Rural) or 100 feet (Urban).

Compound curves are sometimes used for turning roadways at intersections. For turning roadways and intersections a ratio of 2:1 (where the flatter radius precedes the sharper radius in the direction of travel) is acceptable. The arc lengths of compound curves for turning roadways when followed by a curve of one half radius or preceded by a curve of double radius should be as shown in **Table 3 – 9 Length of Compound Curves on Turning Roadways**.

Table 3-9 Length of Compound Curves on Turning Roadways

Radius (feet)	100	150	200	250	300	400	≥ 500
Desirable Arc Length ~~(feet)~~	~~65~~60	70	~~100~~90	120	~~150~~140	180	200
Minimum Arc Length ~~(feet)~~	40	50	~~65~~60	~~85~~80	100	120	~~150~~140

Source: AASHTO Green Book (2018), Table 3-14. Lengths of Circular Arcs for Difference Compound Curve Radii

3.3.4.3 Superelevation

In the design of street and highway curves, it is necessary to establish a proper relationship between curvature of the roadway and design speed. The use of superelevation (rotation of the roadway about its axis) is employed to counteract centrifugal force and allow drivers to comfortably and safely travel through curves at the design speed.

The terms Rural and Urban used in this section reflect the location of the project. In addition to the criteria provided below, additional information regarding superelevation given in the ***FDOT Design Manual***, and ***A Policy on Geometric Design of Highways and Streets (AASHTO, 2018)***~~***A Policy on Geometric Design of Highways and Streets (AASHTO, 2011)***~~, may be considered.

3.3.4.3.1 Rural Highways, Urban Freeways and High Speed Urban Highways

The superelevation rates for high speed (50 mph or greater) roadways are provided in **Table 3 – 10 Superelevation Rates for Rural Highways, Urban Freeways and High-Speed Urban Highways (e_{max} =0.10)**. These rates are based on Method 5 from the ***AASHTO Green Book (2018)***~~***2011 AASHTO Greenbook***~~ using a maximum rate of 0.10 foot per foot of roadway width. **Table 3 – 10** also provides the minimum radius required for normal crown without superelevation.

3.3.4.3.2 Low Speed Urban Roadways

For low speed (45 mph and less) roadways in urban areas, various factors combine to make superelevation difficult, if not impractical, in many built-up areas. Such factors include:

- Wide pavement areas
- Need to meet grade of adjacent property
- Surface drainage considerations
- Frequency of cross streets, alleys, and driveways

Superelevation rates for low speed urban roadways therefore rely more heavily on side friction than rates used for high speed roadways and the maximum superelevation rate is set at 0.05 foot per foot. Separate criteria are provided for low speed Local Roads vs. low speed Arterials and Collectors as follows:

- Low Speed Urban Arterials and Collectors: Superelevation rates for low speed urban arterials and collectors are provided in **Table 3 – 11 Superelevation Rates for Low Speed Arterials and Collectors (e_{max} = 0.05)**. These rates are based on the FDOT's superelevation criteria for low speed arterials and collectors. **Table 3 – 11** also provides the minimum radius required for normal crown without superelevation.
- Low Speed Local Roads: Minimum radii for design superelevation rates for low speed local roads are provided in **Table 3 – 12 Minimum Radii (feet) for Design Superelevation Rates, Low Speed Local Roads (e_{max} = 0.05)**. These rates are based on Method 2 from the ***AASHTO Green Book (2018)***~~2011 AASHTO Greenbook~~. **Table 3 – 12** also provides the minimum radius required for normal crown (-0.02 ft/ft) without superelevation.

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that has not been adopted.

Table 3-10 Superelevation Rates for Rural Highways, Urban Freeways and High Speed Urban Highways (e_{max} = 0.10)

Tabulated Values

Degree of Curve D	Radius R (ft.)	Design Speed (mph)								
		30	35	40	45	50	55	60	65	70
0° 15'	22,918	NC	NC	NC	NC	NC	NC	NC	NC	NC
0° 30'	11,459	NC	NC	NC	NC	NC	NC	RC	RC	RC
0° 45'	7,639	NC	NC	NC	NC	RC	RC	0.023	0.025	0.028
1° 00'	5,730	NC	NC	NC	RC	0.021	0.025	0.030	0.033	0.037
1° 15'	4,584	NC	NC	RC	0.022	0.026	0.031	0.036	0.041	0.046
1° 30'	3,820	NC	RC	0.021	0.026	0.031	0.037	0.043	0.048	0.054
	*R_{NC}									
2° 00'	2,865	RC	0.022	0.028	0.034	0.040	0.048	0.055	0.062	0.070
	*R_{RC}									
2° 30'	2,292	0.021	0.028	0.034	0.041	0.049	0.058	0.067	0.075	0.085
3° 00'	1,910	0.025	0.032	0.040	0.049	0.057	0.067	0.077	0.087	0.096
3° 30'	1,637	0.029	0.037	0.046	0.055	0.065	0.075	0.086	0.095	0.100
4° 00'	1,432	0.033	0.042	0.051	0.061	0.072	0.083	0.093	0.099	Dmax = 3° 30'
5° 00'	1,146	0.040	0.050	0.061	0.072	0.083	0.094	0.098	Dmax = 4 ° 15'	
6° 00'	955	0.046	0.058	0.070	0.082	0.092	0.099	Dmax = 5° 15'		
7° 00'	819	0.053	0.065	0.078	0.089	0.098	Dmax = 6° 30'			
8° 00'	716	0.058	0.071	0.084	0.095	0.100				
9° 00'	637	0.063	0.077	0.089	0.098	Dmax = 8° 15'				
10° 00'	573	0.068	0.082	0.094	0.100					
11° 00'	521	0.072	0.086	0.097	Dmax = 10° 15'					
12° 00'	477	0.076	0.090	0.099						
13° 00'	441	0.080	0.093	0.100						
14° 00'	409	0.083	0.096	Dmax = 13° 15'						
15° 00'	382	0.086	0.098							
16° 00'	358	0.089	0.099							
18° 00'	318	0.093	Dmax = 17° 45'							
20° 00'	286	0.097								
22° 00'	260	0.099								
24° 00'	239	0.100								
		Dmax = 24° 45'								

Break Points	Design Speed (mph)								
	30	35	40	45	50	55	60	65	70
R_{NC}	3349	4384	5560	6878	8337	9949	11709	13164	14714
R_{RC}	2471	3238	4110	5087	6171	7372	8686	9783	10955

e = NC if R ≥ R_{NC} e = RC if R < R_{NC} and R ≥ R_{RC}

NC = Normal Crown (-0.02) RC = Reverse Crown (+0.02)
RNC= Minimum Radius for NC RRC = Minimum Radius for RC
Rates for intermediate D and R's are to be interpolated.

Table 3-11 Superelevation Rates for Low Speed Arterials and Collectors (e_{max} = 0.05)

Tabulated Values					
Degree of Curve D	**Radius R (ft.)**	**Design Speed (mph)**			
		25 - 30	**35**	**40**	**45**
2° 00'	2,865	NC	NC	NC	NC
2° 15'	2,546				
2° 45'	2,083				NC
3° 00'	1,910				RC
3° 45'	1,528			NC	
4° 00'	1,432			RC	
4° 45'	1,206				
5° 00'	1,146		NC		
5° 15'	1,091		RC		
5° 30'	1,042				
5° 45'	996				
6° 00'	955				RC
6° 15'	917				0.022
6° 30'	881				0.024
6° 45'	849				0.027
7° 00'	819	NC			0.030
7° 15'	790	RC			0.033
7° 30'	764				0.037
7° 45'	739				0.041
8° 00'	716			RC	0.045
8° 15'	694			0.022	0.050
8° 30'	674			0.025	**Dmax = 8° 15'**
8° 45'	655			0.027	
9° 00'	637			0.030	
9° 30'	603			0.034	
10° 00'	573			0.040	
10° 30'	546		RC	0.047	
11° 00'	521		0.023	**Dmax = 10° 45'**	
11° 30'	498		0.026		
12° 00'	477		0.030		
13° 00'	441		0.036		
14° 00'	409	RC	0.045		
15° 00'	382	0.023	**Dmax = 14° 15'**		
16° 00'	358	0.027			
17° 00'	337	0.032			
18° 00'	318	0.038			
19° 00'	302	0.043			
20° 00'	286	0.050			
		Dmax = 20° 00'			
NC = Normal Crown (-0.02) RC = Reverse Crown (+0.02) Rates for intermediate D and R's are to be interpolated.					

Table 3-12 Minimum Radii (feet) for Design Superelevation Rates Low Speed Local Roads (e_{max} = 0.05)

e - ft/ft	Design Speed (mph)							
	10	15	20	25	30	35	40	45
0.05	16	41	83	149	240	355	508	675
0.045	16	41	85	152	245	363	520	692
0.04	16	42	86	154	250	371	533	711
0.035	16	42	87	157	255	380	547	730
0.03	16	43	89	160	261	389	561	750
0.025	16	43	90	163	267	398	577	771
0.02	17	44	92	167	273	408	593	794
0.015	17	45	94	170	279	419	610	818
0.01	17	45	95	174	286	430	627	844
0.005	17	46	97	177	293	441	646	871
0	18	47	99	181	300	454	667	900
-0.01	18	48	103	189	316	480	711	964
-0.02	19	50	107	198	333	510	762	~~1038~~1039
-0.03[1]	19	52	111	208	353	544	821	1125
-0.04[1]	20	54	116	219	375	583	889	1227
-0.05[1]	20	56	121	231	400	628	970	1350

Notes:
1. Negative superelevation values beyond -0.02 feet per foot should be used only for unpaved surfaces such as gravel, crushed stone, and earth.

3.3.4.4 Maximum Curvature/Minimum Radius

Where a directional change in alignment is required, every effort should be made to utilize the smallest degree (largest radius) curvature possible. The use of the maximum degree of curvature should be avoided when possible. Design speed maximum degree of curvature or minimum radius for the maximum superelevation rates are provided in **Tables 3 – 10 Superelevation Rates for Rural Highways, Urban Freeways and High-Speed Urban Highways**, **3 – 11 Superelevation Rates for Low Speed Arterials and Collectors**, and **3 –**

12 Minimum Radii (feet) for Design Superelevation Rates Low Speed Local Roads. The use of sharper curvature would call for superelevation beyond the limit considered practical or for operation with tire friction beyond safe or comfortable limits or both. The maximum degree of curvature or minimum radius is a significant value in alignment design.

3.3.4.5 Superelevation Transition (superelevation runoffs plus tangent runoff)

Superelevation runoff is the general term denoting the length of street or highway needed to transition the change in cross slope from a section with the adverse crown removed (level) to the fully superelevated section, or vice versa. Tangent runoff is the general term denoting the length of street or highway needed to accomplish the change in cross slope from a normal cross section to a section with the adverse crown removed, or vice versa.

The standard superelevation transition places 80% of the transition on the tangent and 20% on the curve. In transition sections where the travel lane(s) cross slope is less than 1.5 %, one of the following grade criteria should be applied:

- Maintain a minimum profile grade of 0.5%, or
- Maintain a minimum edge of pavement grade of 0.2% (0.5% for curbed roadways).

When superelevation is required for curves in opposite directions on a common tangent (reverse curves), a suitable distance is required between the curves. This suitable tangent length should be determined as follows:

- 80% of the transition for each curve should be located on the tangent.
- The suitable tangent length is the sum of the two 80% distances, or greater.
- Where alignment constraints dictate a less than desirable tangent length between curves, an adjustment of the 80/20 superelevation transition treatment is allowed (where up to 50% of the transition may be placed on the curve).

Superelevation transition slope rates used to compute transition lengths are provided in **Table 3 –13 Superelevation Transition Slope Rates**. The ***AASHTO Green Book (2018)***~~2011 AASHTO Greenbook~~ provides additional information on superelevation transition design.

The ***FDOT's Standard Plans for Road and Bridge Construction*** provide additional information on superelevation transitions for various sections and methods for determining length of transition.

Table 3-13 Superelevation Transition Slope Rates

Number of Lanes in One Direction	High Speed Roadways Design Speed (mph)				Low Speed Roadways Design Speed (mph)		
	25-40	45-50	55-60	65-70	25-35	40	45
1-Lane & 2-Lane	1:175	1:200	1:225	1:250	1:100	1:125	1:150
3-Lane	---	1:160	1:180	1:200			
4-Lane or more	---	1:150	1:170	1:190			

High Speed Roadways:
1. The length of superelevation transition is to be determined by the relative slope rate between the travel way edge of pavement and the profile grade, except that the minimum length of transition is 100 feet.
2. For additional information on transitions, see the ***Standard Plans***, Index 000-510.

Low Speed Roadways:
1. The length of superelevation transition is to be determined by the relative slope rate between the travel way edge of pavement and the profile grade, except that the minimum length of transition is 50 feet for design speeds 25-35 mph and 75 ft. for design speeds 40-45.
2. A slope rate of 1:125 may be used for 45 mph under restricted conditions.
3. For additional information on transitions, see the ***Standard Plans***, Index 000-511.

Spiral curves may be used to transition from the tangent to the curve. Where the spiral curve is employed, its length is used to make the entire superelevation transition. For additional information on the use of spiral curves, see the ***AASHTO Green Book (2018)***~~***2011 AASHTO Greenbook***~~.

3.3.4.6 Sight Distance on Horizontal Curves

Where there are sight obstructions (such as walls, cut slopes, buildings, and longitudinal barriers) on the inside of curves or the inside of the median lane on divided highways and their removal to increase sight distance is impractical, a design may need adjustment in the normal highway cross section or alignment. With sight distance for the design speed as a control, make the appropriate adjustments to provide adequate stopping sight distance. **Figure 3 – 1 Horizontal Sight Line Offset Distances for Stopping Sight Distance on Horizontal Curves** and **Figure 3 – 2 Diagram Illustrating Components for Determining Horizontal Sight Distance** show the horizontal sight line offsets needed for clear sight areas that satisfy stopping sight distance criteria presented in **Table 3 – 4 Minimum Stopping Sight Distances** for horizontal curves of radii on flat grades.

Figure 3-1 Horizontal Sight Line Offset Distances for Stopping Sight Distance on Horizontal Curves

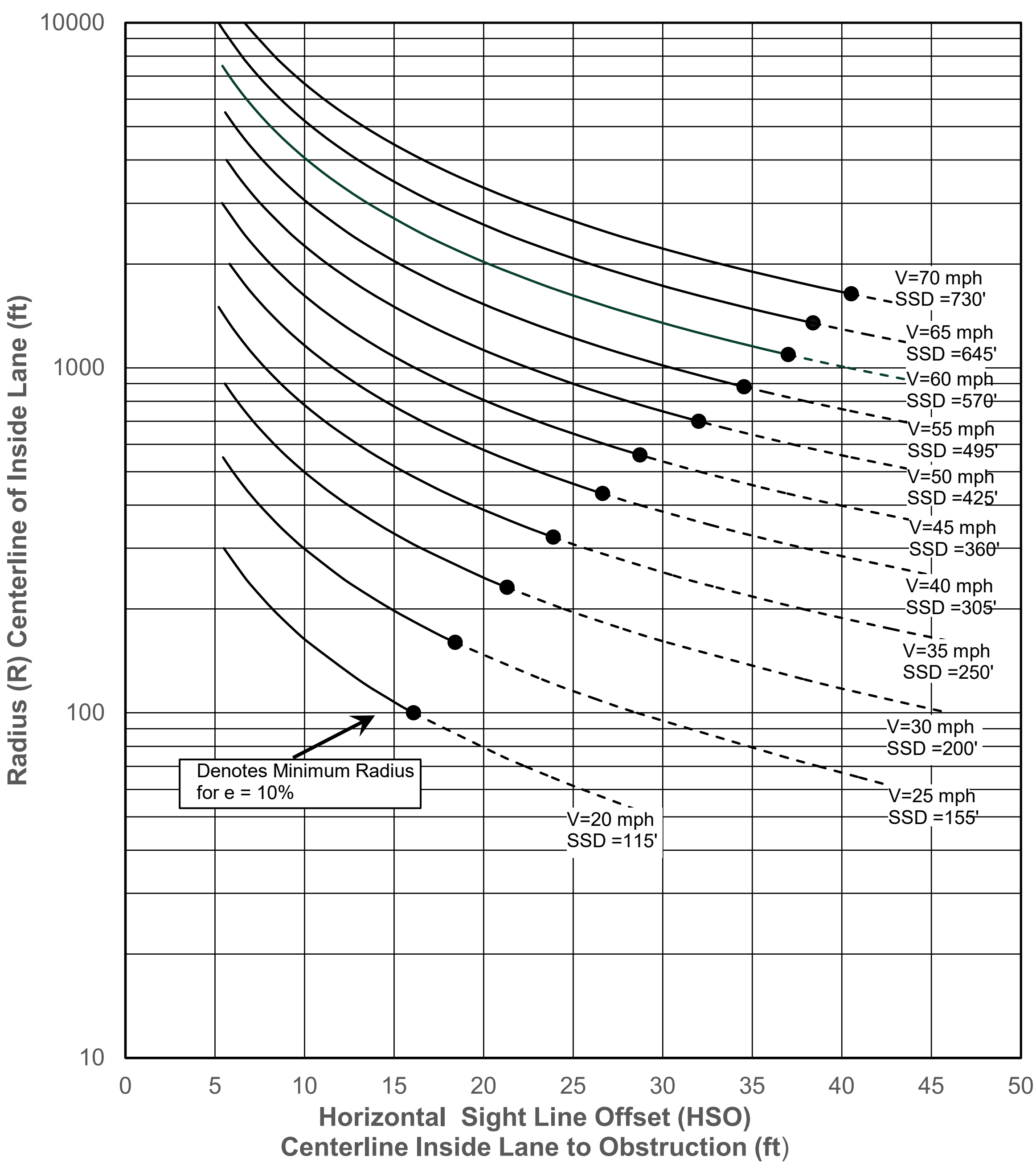
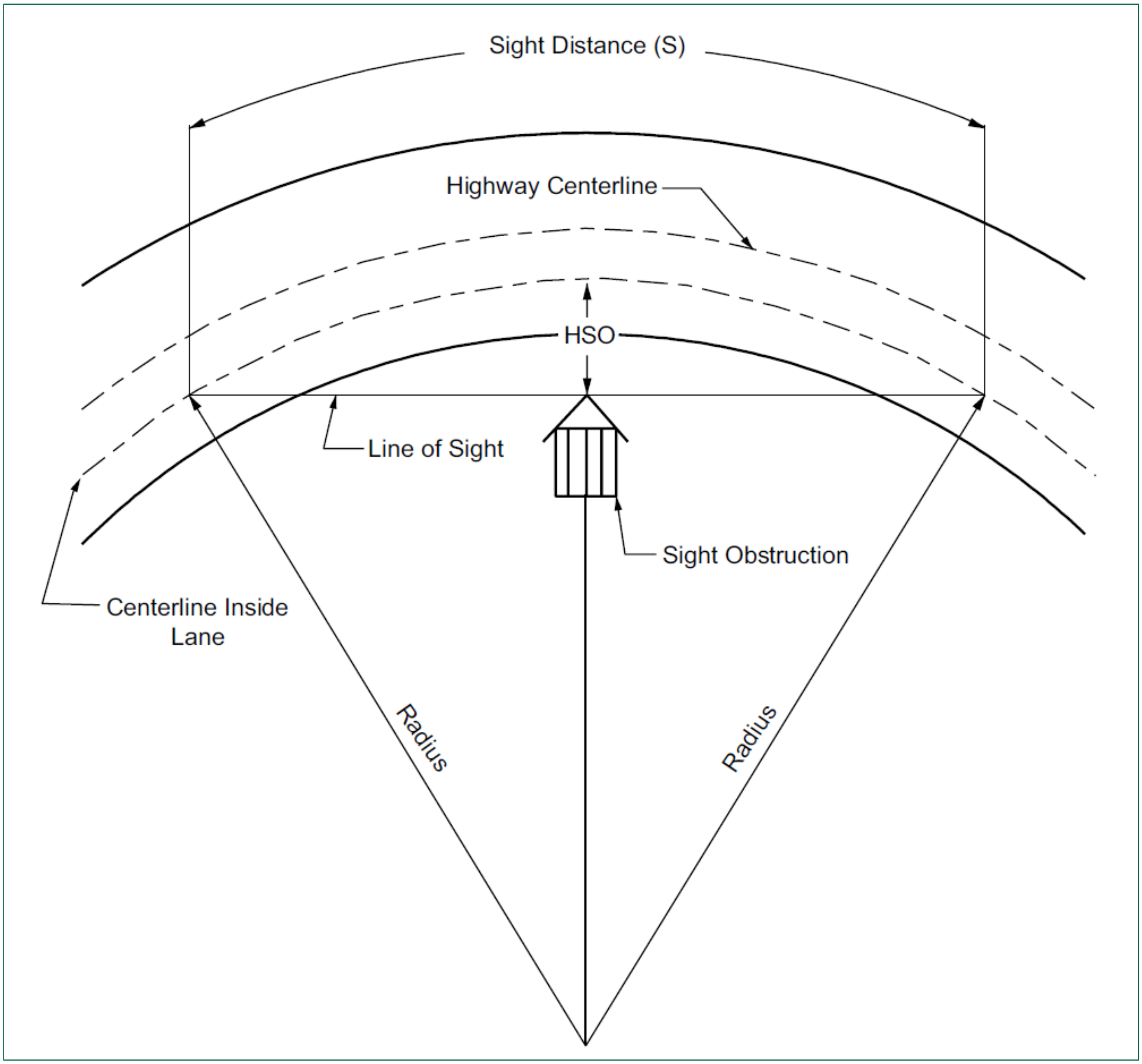


Figure 3-2 Diagram Illustrating Components for Determining Horizontal Sight Distance



HSO – Horizontal Sight Distance

Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Figure 3 – ~~23~~13. Diagram Illustrating Components for Determining Horizontal Sight Distance

Table 3-14 Horizontal Curvature

Lateral Clearance from Edge of Traveled Way to Obstruction For Maximum Curvature (Degrees), Based on Line of Sight On Inside Lane (Lateral Clearance = M Inside Lane – 6') Based on e_{max} = 0.10		
Design Speed (mph)	**Maximum Curvature**	**Clearance (feet)**
20	57° 45’	11
25	36° 15’	13
30	24° 45'	16
35	17° 45'	19
40	13° 30'	21
45	10° 15'	23
50	8° 15'	27
55	6° 30'	29
60	5° 15'	31
65	4° 15'	33
70	3° 30'	35

3.3.4.7 Lane Widening on Curves

The traveled way should be widened on sharp curves due to the increased difficulty for the driver to follow the proper path. Trucks and transit vehicles experience additional difficulty due to the fact that the rear wheels may track considerably inside the front wheels thus requiring additional width. Adjustments to traveled way widths for mainline and turning roadways are given in **Tables 3 – 15~~A~~ Calculated and Design Values for Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)** and **3 – ~~15B~~ 16 Adjustments or Traveled Way Widening Values on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)**. A transition length shall be introduced in changing to an increased/decreased lane width. This transition length shall be proportional to the increase/decrease in traveled way width in a ratio of not less than 50 feet of transition length for each foot of change in lane width.

This is a working document that has not been adopted.

Table 3-15 Calculated and Design Values for Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

Radius of Curve (feet)	Roadway width = 24 feet. Design Speed (mph)							Roadway width = 22 feet. Design Speed (mph)							Roadway width = 20 feet. Design Speed (mph)						
	30	35	40	45	50	55	60	30	35	40	45	50	55	60	30	35	40	45	50	55	60
7000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.7	0.8	0.8	0.9	1.0	1.0	1.7	1.7	1.8	1.8	1.9	2.0	2.0
6500	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.7	1.8	1.8	1.9	2.0	2.0	2.1
6000	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.7	1.8	1.9	1.9	2.0	2.0	2.1
5500	0.0	0.0	0.0	0.0	0.1	0.1	0.2	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.8	1.9	1.9	2.0	2.1	2.1	2.2
5000	0.0	0.0	0.0	0.1	0.1	0.2	0.3	0.9	0.9	1.0	1.1	1.1	1.2	1.3	1.9	1.9	2.0	2.1	2.1	2.2	2.3
4500	0.0	0.0	0.1	0.1	0.2	0.3	0.4	0.9	1.0	1.1	1.1	1.2	1.3	1.4	1.9	2.0	2.1	2.1	2.2	2.3	2.4
4000	0.0	0.1	0.2	0.2	0.3	0.4	0.5	1.0	1.1	1.2	1.2	1.3	1.4	1.5	2.0	2.1	2.2	2.2	2.3	2.4	2.5
3500	0.1	0.2	0.3	0.4	0.5	0.5	0.6	1.1	1.2	1.3	1.4	1.5	1.5	1.6	2.1	2.2	2.3	2.4	2.5	2.5	2.6
3000	0.3	0.4	0.4	0.5	0.6	0.7	0.8	1.3	1.4	1.4	1.5	1.6	1.7	1.8	2.3	2.4	2.4	2.5	2.6	2.7	2.8
2500	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.5	2.6	2.7	2.8	2.9	3.0	3.1
2000	0.7	0.9	1.0	1.1	1.2	1.3	1.4	1.7	1.9	2.0	2.1	2.2	2.3	2.4	2.7	2.9	3.0	3.1	3.2	3.3	3.4
1800	0.9	1.0	1.1	1.3	1.4	1.5	1.6	1.9	2.0	2.1	2.3	2.4	2.5	2.6	2.9	3.0	3.1	3.3	3.4	3.5	3.6
1600	1.1	1.2	1.3	1.5	1.6	1.7	1.8	2.1	2.2	2.3	2.5	2.6	2.7	2.8	3.1	3.2	3.3	3.5	3.6	3.7	3.8
1400	1.3	1.5	1.6	1.7	1.9	2.0	2.1	2.3	2.5	2.6	2.7	2.9	3.0	3.1	3.3	3.5	3.6	3.7	3.9	4.0	4.1
1200	1.7	1.8	1.9	2.1	2.2	2.4	2.5	2.7	2.8	2.9	3.1	3.2	3.4	3.5	3.7	3.8	3.9	4.1	4.2	4.4	4.5
1000	2.1	2.3	2.4	2.6	2.7	2.9	3.0	3.1	3.3	3.4	3.6	3.7	3.9	4.0	4.1	4.3	4.4	4.6	4.7	4.9	5.0
900	2.4	2.6	2.7	2.9	3.1	3.2		3.4	3.6	3.7	3.9	4.1	4.2		4.4	4.6	4.7	4.9	5.1	5.2	
800	2.7	2.9	3.1	3.3	3.5	3.6		3.7	3.9	4.1	4.3	4.5	4.6		4.7	4.9	5.1	5.3	5.5	5.6	
700	3.2	3.4	3.6	3.8	4.0			4.2	4.4	4.6	4.8	5.0			5.2	5.4	5.6	5.8	6.0		
600	3.8	4.0	4.2	4.4	4.6			4.8	5.0	5.2	5.4	5.6			5.8	6.0	6.2	6.4	6.6		
500	4.6	4.9	5.1	5.3				5.6	5.9	6.1	6.3				6.6	6.9	7.1	7.3			
450	5.2	5.4	5.7					6.2	6.4	6.7					7.2	7.4	7.7				
400	5.9	6.1	6.4					6.9	7.1	7.4					7.9	8.1	8.4				
350	6.8	7.0	7.3					7.8	8.0	8.3					8.8	9.0	9.3				
300	7.9	8.2						8.9	9.2						9.9	10.2					
250	9.6							10.6							11.6						
200	12.0							13.0							14.0						

Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Table 3 – 24a~~26b~~ Calculated and Design values for Traveled Way Widening on Open Highway Curves.

Notes:

1. Values shown are for WB-62 design vehicle and represent widening in feet. For other design vehicles, use adjustments in **Table 3-16**.
2. Values less than 2.0 feet may be disregarded, For 3-lane roadways
3. For 3-lane roadways, multiply above values by 1.5.

Table 3-16 Adjustments for Traveled Way Widening Values on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

Radius of Curve (feet)	Design Vehicle			
	SU-30	WB-40	WB-67	WB-67D
7000	-1.2	-1.2	0.1	-0.1
6500	-1.3	-1.2	0.1	-0.1
6000	-1.3	-1.2	0.1	-0.2
5500	-1.3	-1.2	0.1	-0.2
5000	-1.3	-1.3	0.1	-0.2
4500	-1.4	-1.3	0.1	-0.2
4000	-1.4	-1.3	0.1	-0.2
3500	-1.5	-1.4	0.1	-0.3
3000	-1.6	-1.4	0.1	-0.3
2500	-1.7	-1.5	0.2	-0.4
2000	-1.8	-1.6	0.2	-0.5
1800	-1.9	-1.7	0.2	-0.5
1600	-2.0	-1.8	0.2	-0.6
1400	-2.2	-1.9	0.3	-0.6
1200	-2.4	-2.1	0.3	-0.8
1000	-2.7	-2.3	0.4	-0.9
900	-2.8	-2.4	0.4	-1.0
800	-3.1	-2.6	0.5	-1.1
700	-3.4	-2.9	0.6	-1.3
600	-3.8	-3.2	0.7	-1.5
500	-4.3	-3.6	0.8	-1.8
450	-4.7	-3.9	0.9	-2.0
400	-5.2	-4.3	1.0	-2.3
350	-5.8	-4.7	1.1	-26
300	-6.6	-5.4	1.3	-3.0
250	-7.7	-6.3	1.6	-3.6
200	-9.4	-7.6	2.0	-4.6

Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Table 3 - 25~~27~~ Adjustments for Traveled Way Widening Values on Open Highway Curves.

Notes:
1. Adjustments are applied by adding to or subtracting from the values in **Table 3-15**.
2. Adjustments depend only on radius and design vehicle; they are independent of traveled way width and design speed.
3. For 3-lane roadways, multiply above values by 1.5.
4. For 4-lane roadways, multiply above values by 2.0.

3.3.5 Vertical Alignment

3.3.5.1 General Criteria

The selection of vertical alignment should be predicated to a large extent upon the following criteria:

- Obtaining maximum sight distances
- Limiting speed differences (particularly for trucks and buses) by reducing magnitude and length of grades
- A "hidden dip" which would not be apparent to the driver must be avoided.
- Steep grades and sharp crest vertical curves should be avoided at or near intersections.
- Flat grades and long gentle vertical curves should be used whenever possible.

3.3.5.2 Grades

The grades selected for vertical alignment should be as flat as practical, and should not be greater than the value given in **Table 3 – 17 Maximum Grades (in Percent)**.

For streets and highways requiring long upgrades, the maximum grade should be reduced so the speed reduction of slow-moving vehicles (e.g., trucks and buses) is not greater than 10 mph. The critical lengths of grade for these speed reductions are shown in **Figure 3 – 3 Critical Length Versus Upgrade**. Where reduction of grade is not practical, climbing lanes should be provided to meet these speed reduction limitations.

The criteria for a climbing lane and the adjacent shoulder are the same as for any travel lane except that the climbing lane should be clearly designated by the appropriate pavement markings. Entrance to and exit from the climbing lane shall follow the same criteria as other merging traffic lanes; however, the climbing lane should not be terminated until well beyond the crest of the vertical curve. Differences in superelevation should not be sufficient to produce a change in pavement cross slope between the climbing lane and through lane in excess of 0.04 feet per foot.

Recommended minimum gutter grades:

- Rolling terrain - 0.5%
- Flat terrain - 0.3%

Table 3-17 Maximum Grades (in Percent)

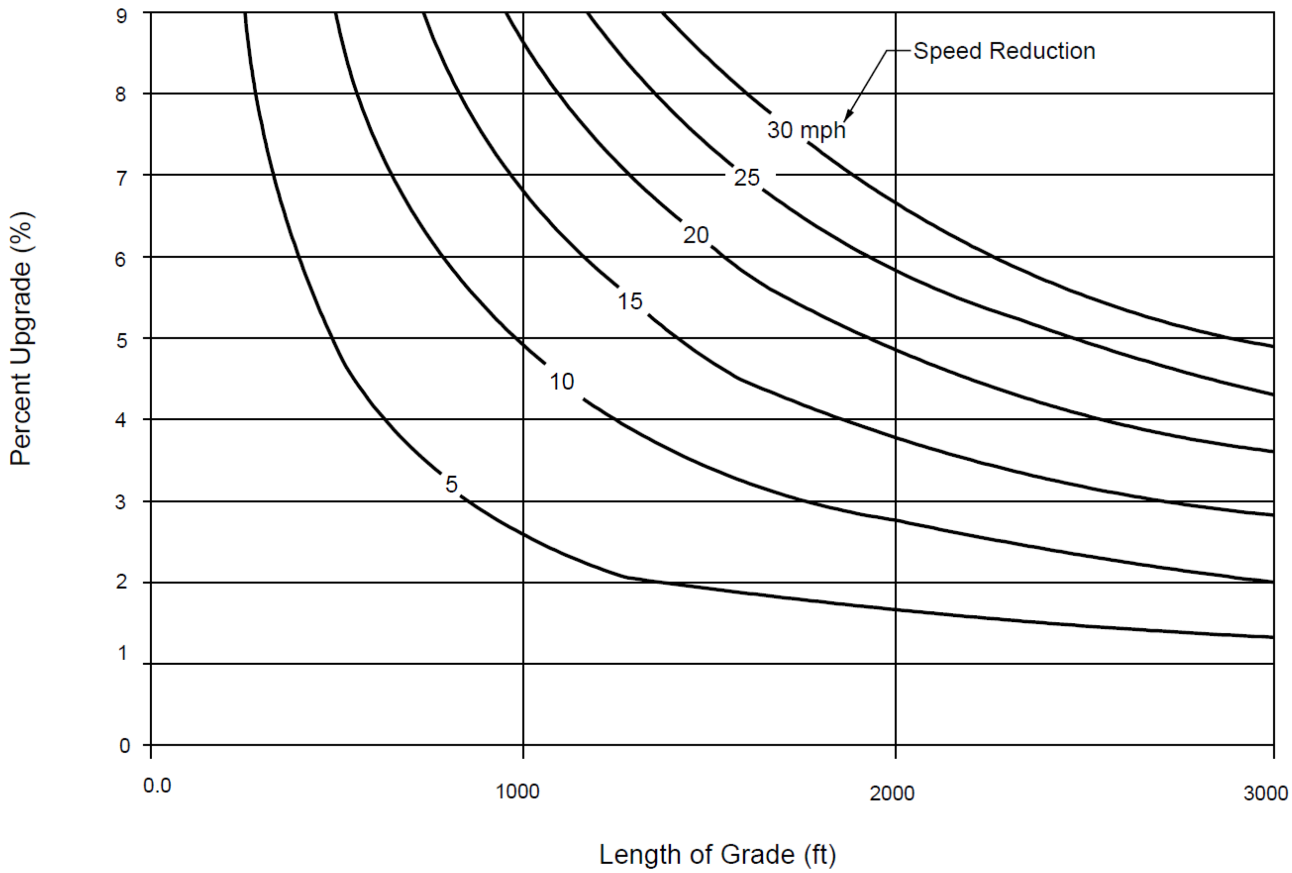
Type of Roadway		Level Terrain											Rolling Terrain										
		Design Speed (mph)											Design Speed (mph)										
		20	25	30	35	40	45	50	55	60	65	70	20	25	30	35	40	45	50	55	60	65	70
Freeway[1]		---	---	---	---	---	---	4	4	3	3	3	---	---	---	---	---	---	5	5	4	4	4
Arterial	Rural	---	---	---	---	5	5	4	4	3	3	3	---	---	---	---	6	6	5	5	4	4	4
	Urban	---	---	8	7	7	6	6	5	5	---	---	---	---	9	8	8	7	7	6	6	---	---
Collector[2]	Rural	7	7	7	7	7	7	6	6	5	---	---	10	10	9	9	8	8	7	7	6	---	---
	Urban	9	9	9	9	9	8	7	7	6	---	---	12	12	11	10	10	9	8	8	7	---	---
Local[3]	Rural	8	7	7	7	7	7	6	6	5	---	---	11	11	10	10	10	9	8	7	6	---	---

Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Tables 5–2, 6–2, 6–7~~8~~, 7–2, 7–4a, 8–1.

Notes:

1. Grades 1% steeper than the value shown may be provided in urban areas with right of way constraints.
2. Short lengths of grade (≤ 500 feet in length), one-way downgrades, and grades on low volume collectors may be up to 2% steeper than the grades shown above.
3. Residential street grade should be as level as practical, consistent with surrounding terrain, and less than 15%. Streets in commercial or industrial areas should have grades less than 8%, and flatter grades should be encouraged.

Figure 3-3 Critical Length Versus Upgrade



Source: ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018), Figure 3-~~28~~21.

Notes:

Critical Lengths of Grade for Design, Assumed Typical Heavy Truck of 200 lb/hp, Entering Speed = 70 mph

3.3.5.3 Vertical Curves

Changes in grade should be connected by a parabolic curve (the vertical offset being proportional to the square of the horizontal distance). Vertical curves are required when the algebraic difference of intersecting grades exceeds the values given in **Table 3 – 18 Maximum Change in Grade Without Using Vertical Curve**. **Table 3 – 19 Rounded K Values for Minimum Lengths Vertical Curves** provides additional information.

The length of vertical curves on a crest, as governed by stopping sight distance, is obtained from **Figure 3 – 4 Length of Crest Vertical Curve (Stopping Sight Distance)**. The minimum length for passing sight distance on crest vertical curves shall be based on the K-values as shown in **Table 3 – 20 Design Controls for Crest Vertical Curves (Passing Sight Distance)**. The minimum length of a sag vertical curve on open road conditions, as governed by vehicle headlight capabilities, is obtained from **Figure 3 - 5 Length of Sag Vertical Curve (Headlight Sight Distance)**.

Wherever feasible, curves longer than the minimum should be considered to improve both aesthetic and safety characteristics.

Table 3-18 Maximum Change in Grade Without Using Vertical Curve

Design Speed (mph)	20	25	30	35	40	45	50	55	60	65	70
Maximum Change in Grade in Percent	1.20	1.10	1.00	0.90	0.80	0.70	0.60	0.50	0.40	0.30	0.20

Table 3-19 Rounded K Values for Minimum Lengths Vertical Curves (Stopping Sight Distance)

Based upon an eye height of 3.50 feet and an object height of 2 feet above the road surface

L = KA

L = Length of Vertical Curve, A = Algebraic Difference of Grades in Percent

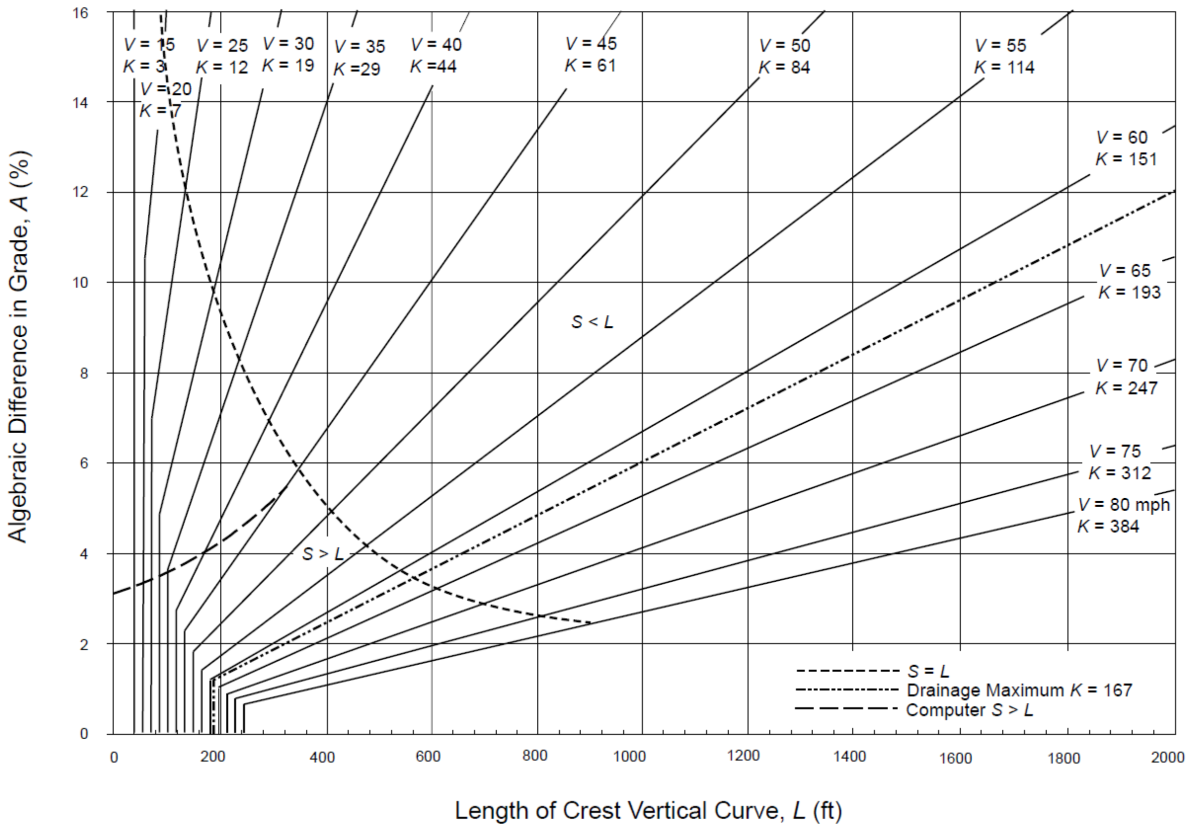
Design Speed (mph)	15	20	25	30	35	40	45	50	55	60	65	70
K Values for Crest Vertical Curves	3	7	12	19	29	44	61	84	114	151	193	247
K Values for Sag Vertical Curves	10	17	26	37	49	64	79	96	115	136	157	181

- The length of vertical curve must never be less than three times the design speed of the highway.
- Curve lengths computed from the formula L = KA should be rounded upward when feasible.
- The minimum lengths of vertical curves to be used on collectors, arterials and freeways are shown in the table below:

Minimum Lengths for Vertical Curves on Collectors, Arterials, and Freeways (feet)

Design Speed (mph)	50	60	70
Crest Vertical Curves (feet)	300	400	500
Sag Vertical Curves (feet)	200	300	400

Figure 3-4 Length of Crest Vertical Curve (Stopping Sight Distance)



Source: Figure 3-~~43~~ 36 Design Controls for Crest Vertical Curves – Open Road Conditions, ~~2011 AASHTO Greenbook~~AASHTO Green Book (2018)

Lengths of crest vertical curves are computed from the formulas:

- When S is less than L, $L=AS^2/2158$
- When S is greater than L, $L=2S – (2158/A)$

A = Algebraic Difference In Grades In Percent

S = Sight Distance

L = Minimum Length of Vertical Curve In Feet

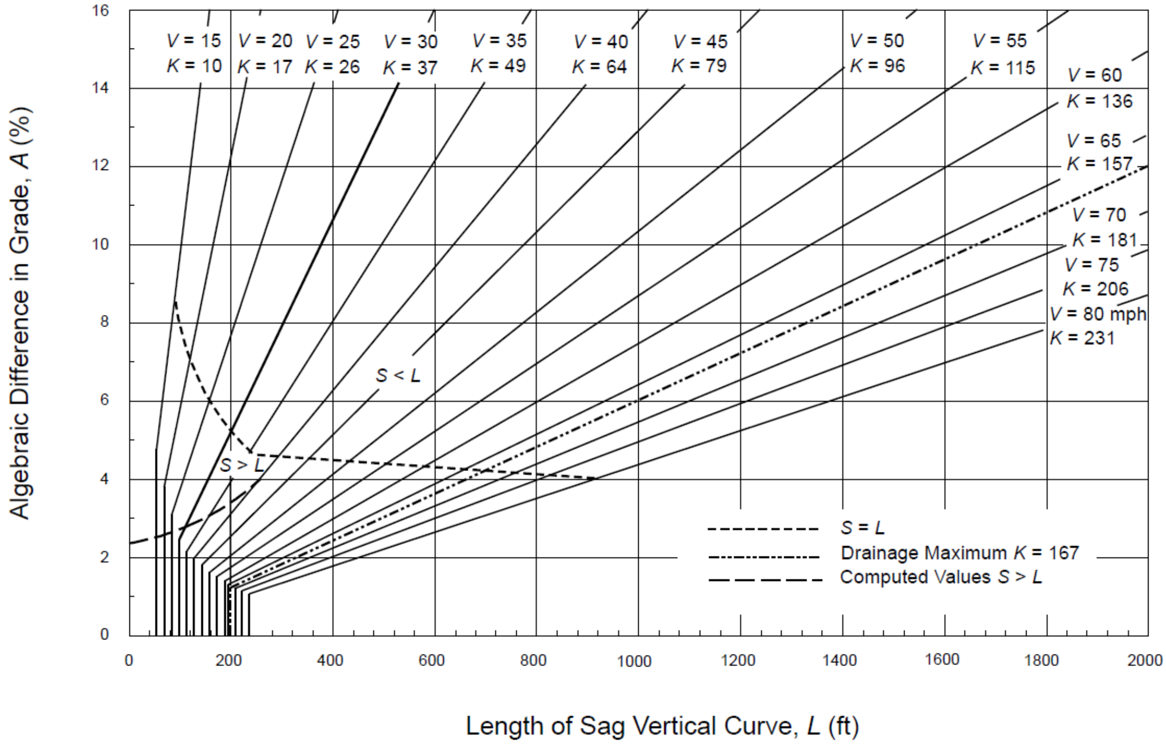
Table 3-20 Design Controls for Crest Vertical Curves (Passing Sight Distance)

Based upon an eye height of 3.50 feet and an object height of **3.5** feet above the road surface.		
L = KA L = Length of Vertical Curve, A = Algebraic Difference of Grades in Percent		
Design Speed (mph)	**Passing Sight Distance (feet)**	**Rate of Vertical Curvature, K[a]**
20	400	57
25	450	72
30	500	89
35	550	108
40	600	129
45	700	175
50	800	229
55	900	289
60	1000	357
65	1100	432
70	1200	514
[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in intersecting grades (A), K = L/A.		

Source: Table 3-36~~35~~ Design Controls for Crest Vertical Curves Based on Passing Sight Distance, ~~2011~~ 2018 AASHTO Greenbook.

For further information on both crest and sag vertical curves, see Section 3.4.6 Vertical Curves of the ***AASHTO Green Book (2018)***~~***AASHTO Greenbook (2011)***~~.

Figure 3-5 Length of Sag Vertical Curve (Open Road Conditions)



Source: Figure 3-~~44~~ 37 Design Controls for Sag Vertical Curves – Open Road Conditions, ~~2011~~ 2018 AASHTO Greenbook.

Lengths of sag vertical curves are computed from the formulas:

- When S is less than L, $L=AS^2/(400 + 3.5S)$
- When S is greater than L, $L=2S - ((400 + 3.5S)/A)$

L = Length of Sag Vertical Curve, feet

A = Algebraic Difference in Grades, percent

S = Light Beam Distance, feet

3.3.6 Alignment Coordination

Horizontal and vertical alignment should not be designed independently. Poor combinations can spoil the good points of a design. Properly coordinated horizontal and vertical alignment can improve appearance, enhance community values, increase safety, and encourage uniform speed. Coordination of horizontal and vertical alignment should begin with preliminary design, during which stage adjustments can be readily made.

Proper combinations of horizontal alignment and profile can be obtained by engineering study and consideration of the following general controls:

- Curvature and grades should be in proper balance. Tangent alignment or flat curvature with steep grades and excessive curvature with flat grades are both poor design. A logical design is a compromise between the two conditions. Wherever feasible the roadway should "roll with" rather than "buck" the terrain.
- Vertical curvature superimposed on horizontal curvature, or vice versa, generally results in a more pleasing facility, but it should be analyzed for effect on driver's view and operation. Changes in profile not in combination with horizontal alignment may result in a series of disconnected humps to the driver for some distance.
- Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. Drivers cannot perceive the horizontal change in alignment, especially at night. This condition can be avoided by setting the horizontal curve so it leads the vertical curve or by making the horizontal curve longer. Suitable design can be made by using design values well above the minimums.
- Sharp horizontal curvature should not be introduced at or near the low point of a pronounced sag vertical curve to prevent an undesirable distorted appearance. Vehicle speeds are often high at the bottom of grades and erratic operation may result, especially at night.
- On divided highways, variation of the median width and the use of independent vertical and horizontal alignment should be considered. Where right of way is available, a superior design without significant additional costs can result from the use of independent alignment.
- Horizontal alignment and profile should be made as flat as possible at interchanges and intersections where sight distance along both highways is important. Sight distances above the minimum are desirable at these locations.
- Alignment should be designed to enhance scenic views for the motorists.
- In residential areas, the alignment should be designed to minimize nuisance to the neighborhood.

3.3.7 Cross Section Elements

The design of the street or highway cross section should be predicated upon the design speed, terrain, adjacent land use, classification, and the type and volume of traffic expected. The cross section selected should be uniform throughout a given length of street or highway without frequent or abrupt changes. See **Chapter 4 – Roadside Design** for design criteria for roadside design, clear zone, lateral offset, and roadside ditches located within the clear zone.

3.3.7.1 Number of Lanes

The number of travel lanes is determined by several interrelated factors such as capacity, level of service, and service volume. Further information on determining the optimum number of travel lanes can be found in ***A Policy on Geometric Design of Highways and Streets (AASHTO, 2018)***~~***A Policy on Geometric Design of Highways and Streets (AASHTO, 2011)***~~, and the ***Highway Capacity Manual (TRB, 2010)***.

3.3.7.2 Pavement

The paved surface of roadways shall be designed and constructed in accordance with the requirements set forth in **Chapter 5 - Pavement Design and Construction**.

3.3.7.2.1 Pavement Width

Minimum lane widths for travel lanes, speed change lanes, turn lanes and passing lanes are provided in **Table 3 – 21 Minimum Lane Widths**. The table applies to both divided and undivided facilities. For Information on parking lanes, see **Section 3.3.7.8** Parking of this Chapter.

On existing multilane curbed streets where there is insufficient space for a separate bicycle lane, consideration should be given to using unequal-width lanes. In such cases, the wider lane is located on the outside (right). This provides more space for large vehicles that usually occupy that lane, provides more space for bicycles, and allows drivers to keep their vehicles at a greater distance from the right edge. See **Chapter 9 – Bicycle Facilities**.

Table 3-21 Minimum Lane Widths

Facility		ADT (vpd)	Design Speed (mph)	Lane Width – (feet)		
				Travel Lanes[1]	Turn Lanes[6] (LT/RT/MD)	Passing Lanes
Freeway	Rural	All	All	12	--	--
	Urban	All	All	12	--	--
Arterial	Rural	All	All	12[9]	12[9]	12[9]
	Urban	All	≥ 50	12	12	12
		All	≤ 45	11[3, 4]	11[3, 4, 7]	11[3, 4]
Collector	Rural	> 1500	All	12[9]	12[9]	12[9]
		401 to 1500	All	11[3, 4]	11[3, 4]	--
		≤ 400	≥ 50	11	11[7]	--
			≤ 45	10	10	--
	Urban	All	All	11[2, 3, 4]	11[2, 7]	--
Local	Rural	> 1500	All	12[9]	12[9]	12[9]
		401 to 1500	All	11[3, 4]	11[3, 4]	--
		≤ 400	≥ 55	11[3]	11[3, 4]	--
			45 to 50	10	10	--
			≤ 40	9	9	--
	Urban	All	All	10[2, 5]	10[8]	--

Notes:

1. A minimum traveled way width equal to the width of two adjacent travel lanes (one way or two way) shall be provided on all rural facilities.
2. In industrial areas and where truck volumes are significant, 12’ lanes should be provided, but may be reduced to 11’ where right of way is constrained.
3. In constrained areas where truck volumes are low and design speeds are ≤ 35 mph, 10’ lanes may be used.
4. On roadways with a transit route, a minimum of 11’ outside lane width is required.
5. In residential areas where right of way is severely limited, 9’ may be used.
6. Turn lane width in raised or grass medians shall not exceed 14’. Two-way left turn lanes should be 11 – 14’ wide and may only be used on 3- and 5-lane typical sections with design speeds ≤ 40 mph. On projects with right of way constraints, the minimum width may be reduced to 10’. Two-way left turn lanes shall include sections of raised or restrictive median for pedestrian refuge.
7. Turn Lane width should be same as Travel Lane width. May be reduced to 10’ where right of way is constrained.
8. Turn Lane width should be same as Travel Lane width. May be reduced to 9’ where truck volumes are low.
9. For design speeds below 50 mph, lane widths of 11 feet are acceptable.

3.3.7.2.2 Traveled Way Cross Slope (not in superelevation)

The selection of traveled way cross slope should be a compromise between meeting the drainage requirements and providing for smooth vehicle operation. The recommended traveled way cross slope is 0.02 feet per foot. When three lanes in each direction are necessary, the outside lane should have a cross slope of 0.03 feet per foot. The cross slope shall not be less than 0.015 feet per foot or greater than 0.04 feet per foot. The change in cross slope between adjacent through travel lanes should not exceed 0.04 feet per foot.

3.3.7.3 Shoulders

The primary functions of a shoulder are to provide emergency parking for disabled vehicles and an alternate path for vehicles during avoidance or other emergency maneuvers. In order to fulfill these functions satisfactorily, the shoulder should have adequate stability and surface characteristics. The design and construction of shoulders shall be in accordance with the requirements given in **Chapter 5 - Pavement Design and Construction**.

Shoulders should be provided on all streets and highways incorporating open drainage. The absence of a contiguous emergency travel or storage lane is not only undesirable from a safety standpoint, but also is disadvantageous from an operations viewpoint. Disabled vehicles that must stop in a through lane impose a severe safety hazard and produce a dramatic reduction in traffic flow. Shoulders should be free of abrupt changes in slope, discontinuities, soft ground, or other hazards that would prevent the driver from retaining or regaining vehicle control.

Paved outside shoulders are required for rural high speed multilane highways and freeways. They provide added safety to the motorist, public transit and pedestrians, for accommodation of bicyclists, reduced shoulder maintenance costs, and improved drainage.

3.3.7.3.1 Shoulder Width

A shoulder is the portion of the roadway contiguous with the traveled way that accommodates stopped vehicles, emergency use, and provides lateral support of subbase, base and surface courses. In some cases, the shoulder may also accommodate pedestrians or bicyclists. Shoulders may be surfaced either full or partial width and include turf, gravel, shell, and asphalt or concrete pavements.

The minimum width of outside and median shoulders is provided in **Table 3 – 22 Minimum Shoulder Widths for Flush Shoulder Highways**. Shoulders for two-lane, two-way highways are based upon traffic volumes. Shoulder widths for multi-lane highways are based upon the number of travel lanes in each direction. Where bicyclists or pedestrians are to be accommodated on the shoulder, a minimum usable width of 4 feet is required (5 feet if adjacent to a barrier). On approaches to narrow bridges where the paved shoulder is reduced, the ***FDOT's Standard Plans*** provide information on signing and marking the approaching shoulder.

Table 3-22 Minimum Shoulder Widths for Flush Shoulder Highways

<table>
<tr><th colspan="5">Two Lane Undivided</th></tr>
<tr><td rowspan="2">Design Speed (mph)</td><td colspan="4">Average Daily Traffic (2 – Way)</td></tr>
<tr><td>0 - ≤400</td><td colspan="2">401 - 750</td><td>>750</td></tr>
<tr><td>All</td><td>2 feet</td><td colspan="2">6 feet</td><td>8 feet</td></tr>
<tr><th colspan="5">Multilane Divided</th></tr>
<tr><td rowspan="3">Number of Lanes Each Direction</td><td colspan="4">Shoulder Width (feet)</td></tr>
<tr><td colspan="2">Outside</td><td colspan="2">Median</td></tr>
<tr><td>Roadway</td><td>Bridge</td><td>Roadway</td><td>Bridge</td></tr>
<tr><td>2</td><td>8 (min.)</td><td>8</td><td>4 (min.)</td><td>4</td></tr>
<tr><td>3 or more</td><td>10 (min.)</td><td>10</td><td>6 (min.)</td><td>6</td></tr>
</table>

3.3.7.3.2 Shoulder Cross Slope

The shoulder serves as a continuation of the drainage system; therefore, the shoulder cross slope should be somewhat greater than the adjacent traffic lane. The cross slope of shoulders should be within the range given in **Table 3 – 23 Shoulder Cross Slope**.

Table 3-23 Shoulder Cross Slope (Percent)

<table>
<tr><th colspan="3">Shoulder Type</th></tr>
<tr><td>Paved</td><td>Gravel or Crushed Rock</td><td>Turf</td></tr>
<tr><td>2 to 6%</td><td>4 to 6%</td><td>6 to 8%</td></tr>
<tr><td colspan="3">Notes:
1. Existing shoulder cross slope (paved and unpaved) ≤ 12% may remain.</td></tr>
</table>

Source: ~~2011 AASHTO Greenbook~~AASHTO Green book (2018), Section 4.4.3 Shoulder Cross Sections.

Whenever possible, shoulders should be sloped away from the traveled way to aid in their drainage. The combination of shoulder cross slope and texture should be sufficient to promote rapid drainage and to avoid retention of surface water. The maximum algebraic difference between the traveled way and adjacent shoulder should not be greater than 0.07 feet per foot. Shoulders on the outside of superelevated curves should be rounded (vertical curve) to avoid an excessive break in cross slope and to divert a portion of the drainage away from the adjacent traveled way.

3.3.7.4 Sidewalks and Shared Use Paths

The design of sidewalks is affected by many factors, including traffic characteristics, pedestrian volume, roadway type, and other design elements**. Chapter 8 - Pedestrian Facilities** and **Chapter 9 – Bicycle Facilities** of this Manual and ***A Policy on Geometric Design of Highways and Streets (AASHTO, 2018)***~~***A Policy on Geometric Design of Highways and Streets (AASHTO, 2011)***~~, present the various factors that influence the design of sidewalks and other pedestrian facilities.

Sidewalks and/or shared use paths should be constructed in conjunction with new construction and major reconstruction in or within one mile of an urban area. As a general rule, sidewalks should be constructed on both sides of the roadway. Exceptions may be made where physical barriers (e.g., a canal paralleling one side of the roadway) would substantially reduce the expectation of pedestrian use of one side of the roadway. Also, if only one side is possible, sidewalks should be available on the same side of the road as transit stops or other pedestrian generators.

The decision to construct a sidewalk or shared use path in a rural area should be based on engineering judgment, after observation of existing pedestrian traffic or expectation of additional demand.

Sidewalks and shared use paths shall be constructed as defined in this Manual. **Chapter 8 – Pedestrian Facilities**, **Chapter 10 – Maintenance and Resurfacing** and **Section 3.3.10.1.3 – Sidewalks and Curb Ramps** of this chapter provide additional detailed information. **AASHTO's Guide for the Planning, Design and Operation of Pedestrian Facilities (2004), and Section 4.17.1 Sidewalks** of ***AASHTO's Policy on Geometric Design of Highways and Streets (2011)*** provide additional information.

The ***Highway Capacity Manual, Volume 3, Chapter 23, Off-Street Pedestrian and Bicycle Facilities (2010)*** includes further information on how optimal widths can be determined.

3.3.7.5 Medians

Median separation of opposing traffic lanes provides a beneficial safety feature and should be used wherever feasible. Separation of the opposing traffic also reduces the problem of headlight glare, thus improving safety and comfort for night driving. When sufficient width of medians is available, some landscaping is also possible.

The use of medians often aids in the provision of drainage for the roadway surface, particularly for highways with six or more traffic lanes. The median also provides a vehicle refuge area, improves the safety of pedestrian crossings, provides a logical location for left turn auxiliary lanes, and provides the means for future addition of traffic lanes and mass transit. In many situations, the median strip aids in roadway delineation and the overall highway aesthetics.

Median separation is required on the following streets and highways:

- Freeways
- All streets and highways, rural and urban, with 4 or more travel lanes and with a design speed of 40 mph or greater

Median separation is desirable on all other multi-lane roadways to enhance pedestrian crossings.

The nature and degree of median separation required is dependent upon the design speed, traffic volume, adjacent land use, and the frequency of access. There are basically two approaches to median separation. The first is the use of horizontal separation of opposing lanes to reduce the probability of vehicles crossing the median into incoming traffic. The second method is to attempt to limit crossovers by introducing a positive median barrier structure.

In rural areas, the use of wide medians is not only aesthetically pleasing, but is often more economical than barriers. In urban areas where space and/or economic constraints are severe, the use of barriers is permitted to fulfill the requirements for median separation.

Uncurbed medians should be free of abrupt changes in slope, discontinuities, soft ground, or other hazards that would prevent the driver from retaining or regaining control of the vehicle. Consideration should be given to increasing the width and decreasing the slope of medians on horizontal curves. The requirements for a hazard free median environment are given in **Chapter 4 - Roadside Design**, and shall be followed in the design and construction of medians.

3.3.7.5.1 Type of Median

A wide, gently depressed median is the preferred design. This type allows a reasonable vehicle recovery area and aids in the drainage of the adjacent shoulders and travel lanes. Where space and drainage limitations are severe, narrower medians, flush with the roadway, or raised medians, are permitted. Raised medians should be used to support pedestrian crossings of multi-laned streets and highways.

3.3.7.5.2 Median Width

The median width is defined as the horizontal distance between the inside (median) edge of travel lanes of the opposing roadways. The selection of the median width for a given type of street or highway is primarily dependent on design speed and traffic volume. Since the probability of crossover crashes is decreased by increasing the separation, medians should be as wide as practicable. Median widths in excess of 30 feet to 35 feet reduce the problem of disabling headlight glare from opposing traffic.

The minimum permitted widths of freeway medians are given in **Table 3 – 24 Minimum Median Width**. Where the expected traffic volume is heavy, the widths should be increased over these minimum values. Median barriers shall be used on freeways when these minimum values are not attainable.

The minimum permitted median widths for multi-lane rural highways are also given in **Table 3 – 24 Minimum Median Width**. On urban streets, the median widths shall not be less than the values given in **Table 3 – 24**. Where median openings or access points are frequent, the median width should be increased.

The minimum median widths given in these Tables may have to be increased to meet the requirements for cross slopes, drainage, and turning movements (**3.3.9 Intersection Design**, this chapter). The median area should also include adequate additional width to allow for expected additions of through lanes and left turn lanes. Where the median width is sufficient to produce essentially two separate, independent roadways, the left side of each roadway shall meet the requirements for roadside clear zone. Changes in the median width should be accomplished by gently flowing horizontal alignment of one or both of the separate roadways.

Table 3-24 Minimum Median Width

Type of Facility	Width (feet)
Freeways	
Freeways, Without Barrier	---
Design Speed ≥ 60 mph	60
Design Speed < 60 mph	40
All, With Barrier, All Design Speeds	26[1]
Arterial and Collectors	
Design Speed ≥ 50 mph	40
Design Speed ≤ 45 mph	22[2]
Paved and Painted for Left Turns	See **Table 3 – 21 Minimum Lane Widths**

Median width is the distance between the inside (median) edge of the travel lane of each roadway.

Notes:
1. Based on 2 ft. wide, concrete median barrier and 12 ft. shoulder.
2. On projects where right of way is constrained, the minimum width may be reduced to 19.5 ft. for design speeds = 45 mph, and to 15.5 ft. for design speeds ≤ 40 mph.

3.3.7.5.3 Median Slopes

A vehicle should be able to traverse a median without turning over and with sufficient smoothness to allow the driver a reasonable chance to control the vehicle. The transition between the median slope and the shoulder (or pavement) slope should be smooth, gently rounded, and free from discontinuities.

The median cross slope should not be steeper than 1:6 (preferably not steeper than 1:10). The depth of depressed medians may be controlled by drainage requirements. Increasing the width of the median, rather than increasing the cross slope, is the proper method for developing the required median depth.

Longitudinal slopes (median profile parallel to the roadway) should be shallow and gently rounded at intersections of grade. The longitudinal slope, relative to the roadway slope, shall not exceed a ratio of 1:10 and preferably 1:20. The change in longitudinal slope shall not exceed 1:8 (change in grade of 12.5 %).

3.3.7.5.4 Median Barriers

See **Chapter 4 – Roadside Design** for criteria on median barriers. The **AASHTO Roadside Design Guide** provides additional information and guidelines on the use of median barriers.

3.3.7.6 Islands

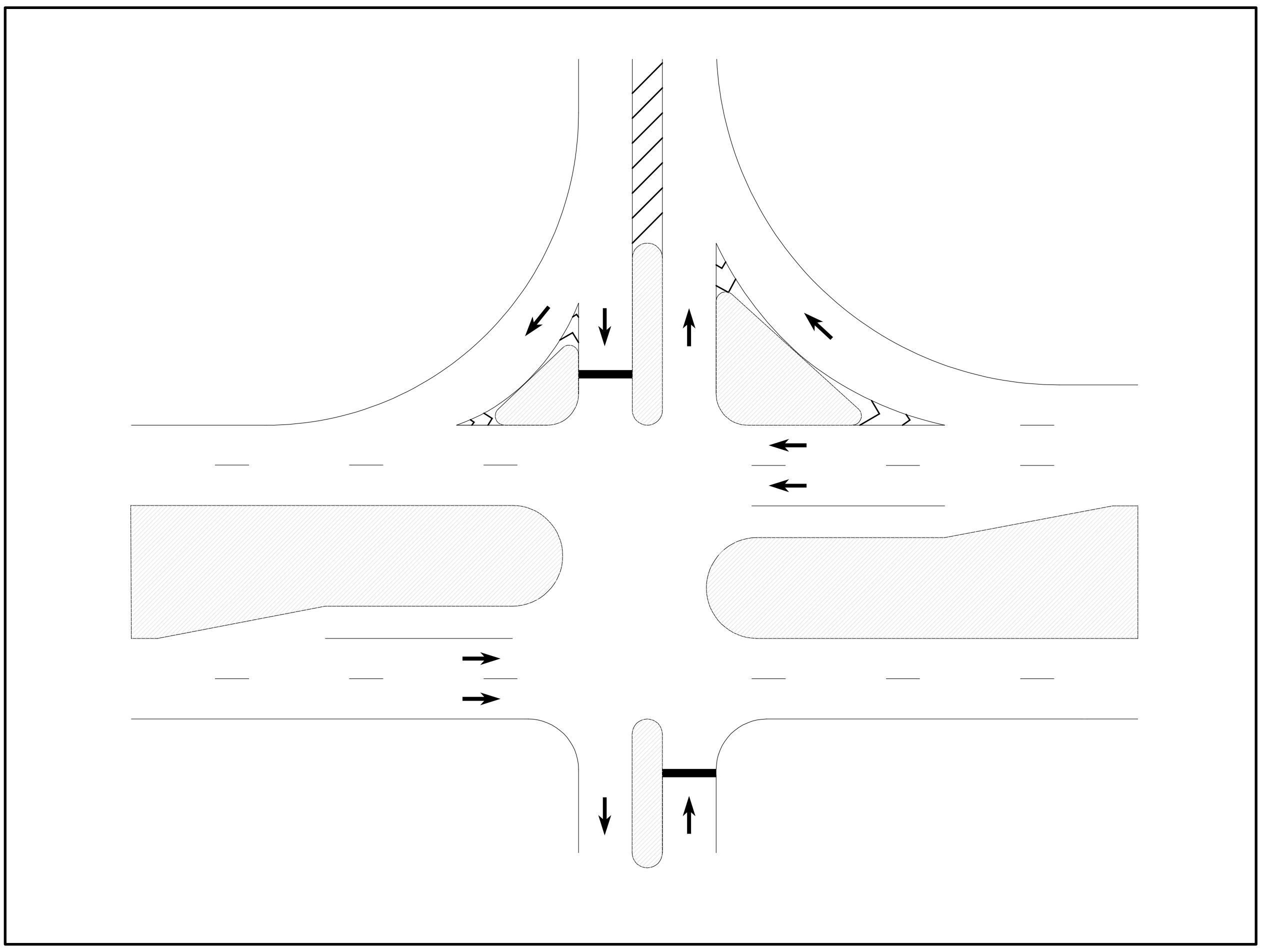
An island is a defined area between traffic lanes used for control of vehicle movements. Most islands combine two or more of these primary functions:

- Channelization — To control and direct traffic movement, usually turning.
- Division — To divide opposing or same direction traffic streams, usually through movements; and
- Refuge — To provide refuge for pedestrians.

Islands generally are either elongated or triangular in shape and situated in areas unused for vehicle paths. Islands should be located and designed to offer little obstruction to vehicles and be commanding enough that motorists will not drive over them. The placement of mast arms in channelizing islands is discouraged. Mast arms are not permitted in median islands.

The dimensions and details depend on the intersection design as illustrated in **Figure 3 – 6 General Types and Shapes of Islands and Medians**. They should conform to the general principles that follow.

Figure 3-6 General Types and Shapes of Islands and Medians



Curbed islands are sometimes difficult to see at night. Where curbed islands are used, the intersection should have fixed–source lighting or appropriate delineation. Under certain conditions, painted, flush medians and islands or traversable type medians may be preferable to the raised curb type islands. These conditions include the following:

- Lightly developed areas that will not be considered for access management.
- Intersections where approach speeds are relatively high.
- Areas where there is little pedestrian traffic.
- Areas where fixed-source lighting is not provided.
- Median or corner islands where signals, signs, or luminaire supports are not needed; and
- Areas where extensive development exists and may demand left-turn lanes into many entrances.

Painted islands may be used at the traveled way edge. At some intersections, both curbed and painted islands may be desirable. All pavement markings should be reflectorized. The use of thermoplastic striping, raised dots, spaced and raised retroreflective markers, and other forms of long-life markings also may be desirable. See **Section 9.6.3** of the ***AASHTO Green Book (2018)***~~***2011 AASHTO Greenbook***~~ and the ***MUTCD, Part 3*** for additional information on the design and marking of islands.

The central area of large channelizing islands in most cases has a turf or other vegetative cover. As space and the overall character of the highway determine, low plant material may be included, but it should not obstruct sight distance. Ground cover or plant growth, such as turf, vines, and shrubs, can be used for channelizing islands and provides excellent contrast with the paved areas, assuming the ground cover is cost-effective and can be properly maintained. The ***FDOT Design Manual, Chapter 212 Intersections*** provides additional information on designing landscaping in medians or at intersections.

Small, curbed islands may be mounded, but where pavement cross slopes are outward, large islands should be depressed to avoid draining water across the pavement. For small, curbed islands and in areas where growing conditions are not favorable, some type of paved surface may be used on the island.

Careful consideration should be given to the location and type of plantings. Plantings, particularly in narrow islands, may create problems for maintenance activities. Plantings and other landscaping features in channelization areas may constitute roadside obstacles and should be consistent with the requirements in **Section 3.3.9.2 Sight Distance**. The ***AASHTO Roadside Design Guide (2011)*** provides additional information on landscaping of islands.

3.3.7.6.1 Channelizing Islands

Channelizing islands may be of many shapes and sizes, depending on the conditions and dimensions of the intersection. A common form is the corner triangular shape that separates right-turning traffic from through traffic. Central islands may serve as a guide around which turning vehicles operate.

Channelizing islands should be placed so that the proper course of travel is immediately obvious, easy to follow, and of unquestionable continuity. Where islands separate turning traffic from through traffic, the radii of curved portions should equal or exceed the minimum for the turning speeds expected. Curbed islands generally should not be used in rural areas and at isolated locations unless the intersection is lighted and curbs are delineated.

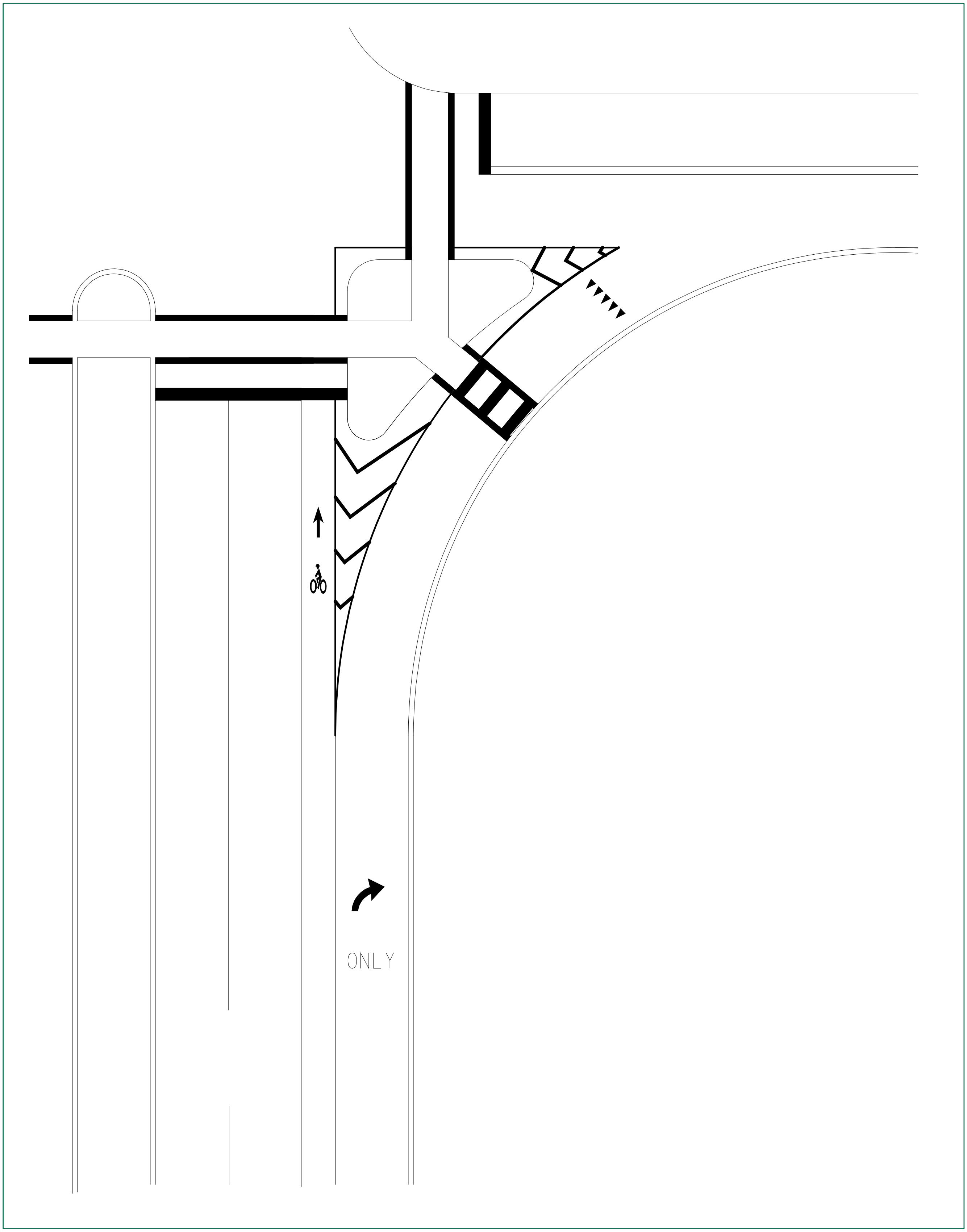
Islands should be sufficiently large to command attention, with 100 ft^2 preferred. The smallest curbed corner island should have an area of at least 50 ft^2 for urban and 75 ft^2 for rural intersections. A corner triangular island should be at least 15 feet on a side (12 ft. minimum) after the rounding of corners.

While mast arms are discouraged in channelizing islands, when they are used the minimum lateral offset as shown in **Chapter 4, Roadside Design Table 4 – 2 Lateral Offset** shall be provided. Mast arm foundation diameters vary from 3.5 feet to 5.0 feet. The minimum lateral offset for 45 mph and less should be based on minimum offset to a hazard from curb face – 4 feet standard, 1.5 feet absolute minimum.

Details of curbed corner island designs used in conjunction with turning roadways are shown in Figures **3 – ~~6~~ 7 Channelization Island for Pedestrian Crossings (Curbed)**, **3 – ~~7~~ 8 Details of Corner Island for Turning Roadways (Curbed)** and **3 – ~~8~~ 9 Details of Corner Island for Turning Roadways (Flush Shoulder)**. The approach corner of each curbed island is designed with an approach nose treatment.

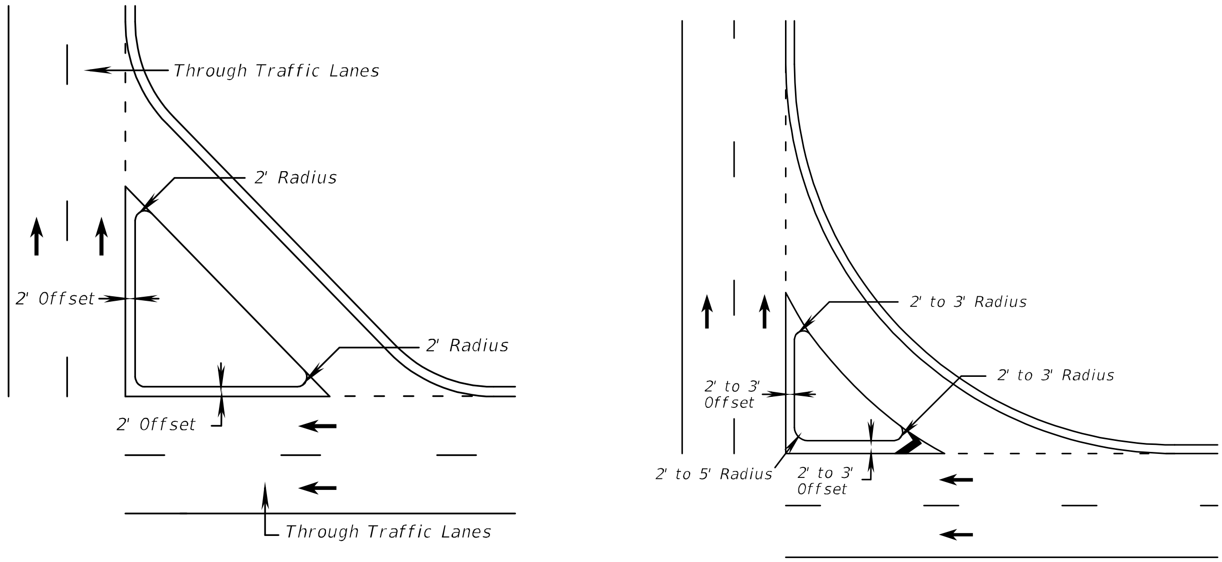
Further information on the pavement markings that can be used with islands can be found in the FDOT's ***Standard Plans, Index 711-001***.

Figure 3-7 Channelization Island for Pedestrian Crossings (Curbed)



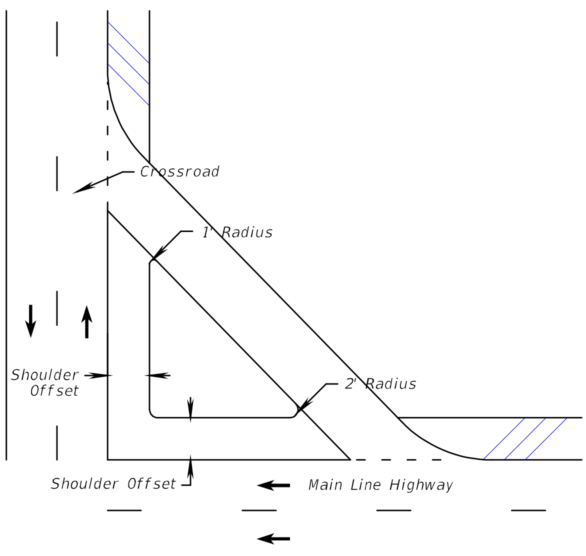
This is a working document that has not been adopted.

Figure 3-8 Details of Corner Island for Turning Roadways (Curbed)



Small ***Intermediate***

Figure 3-9 Details of Corner Island for Turning Roadways (Flush Shoulder)



Small

3.3.7.6.2 Divisional Islands

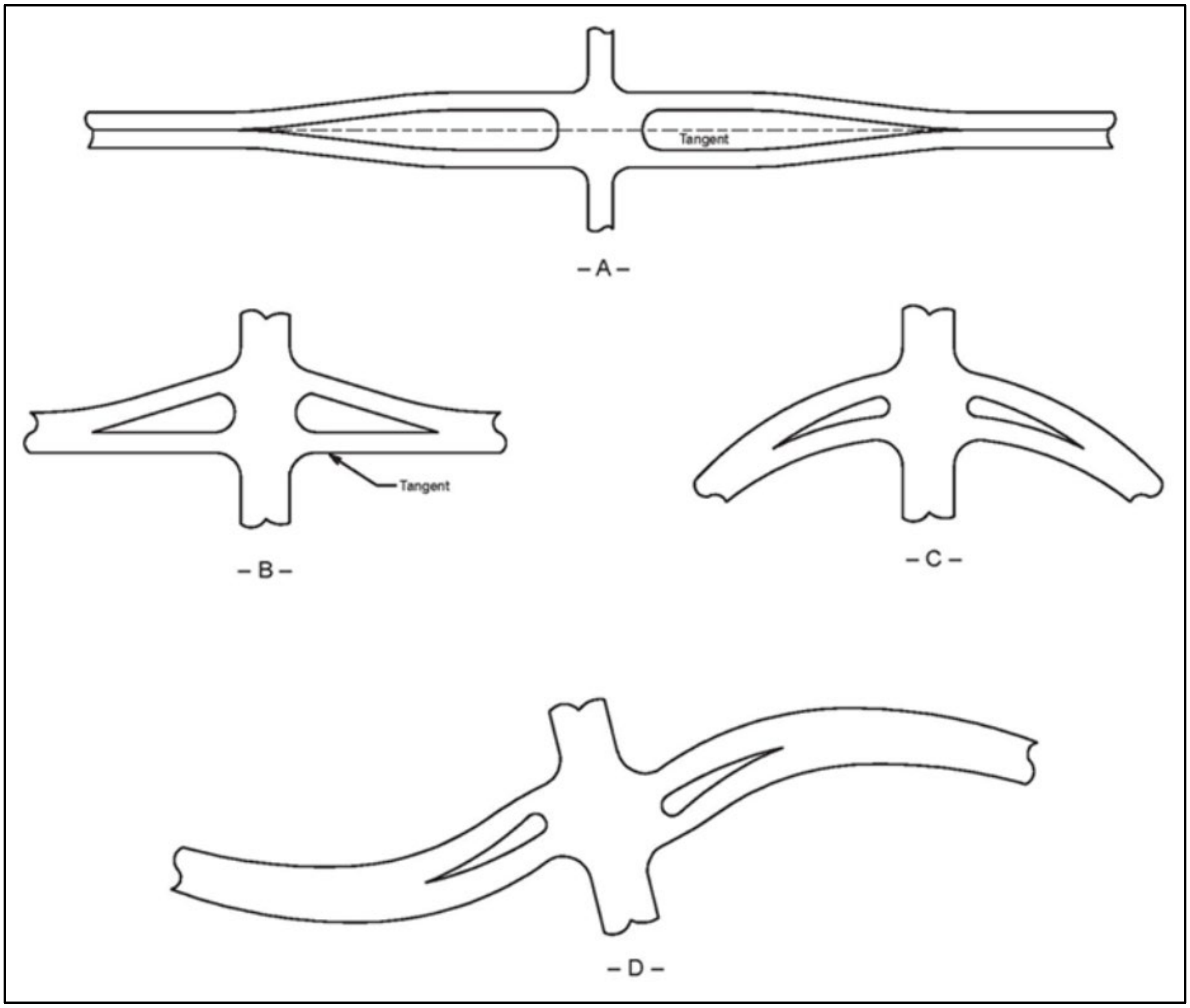
Divisional islands often are introduced on undivided highways at intersections. They alert drivers to the crossroad ahead and regulate traffic through the intersection. These islands are particularly advantageous in controlling left turns at skewed intersections and at locations where separate roadways are provided for right-turning traffic.

Widening a roadway to include a divisional island should be done in such a manner that the proper paths to follow are unmistakably evident to drivers. The alignment should require no appreciable conscious effort in vehicle steering.

Elongated or divisional islands should be not less than 4 feet wide and 20 to 25 feet long. In general, introducing curbed divisional islands at isolated intersections on high-speed highways is undesirable unless special attention is directed to providing high visibility for the islands. Curbed divisional islands introduced at isolated intersections on high-speed highways should be 100 feet or more in length. When situated in the vicinity of a high point in the roadway profile or at or near the beginning of a horizontal curve, the approach end of the curbed island should be extended to be clearly visible to approaching drivers.

Where an island is introduced at an intersection to separate opposing traffic on a four-lane road or on a major two-lane highway carrying high volumes, two full lanes should be provided on each side of the dividing island (particularly where future conversion to a wider highway is likely). In other instances, narrower roadways may be used. For moderate volumes, roadway widths shown under Case II (one-lane, one-way operation with provision for passing a stalled vehicle) in **Table 3 – 38 Derived Pavement Widths for Turning Roadways for Different Design Vehicles** are appropriate. For light volumes and where small islands are needed, widths on each side of the island corresponding to Case I in **Table 3 – ~~35~~ 38**may be used.

Figure 3-10 Alignment for Divisional Islands at Intersections



3.3.7.6.3 Refuge Islands

A refuge island for pedestrians at or near a crosswalk or shared use path crossing aids pedestrians and bicyclists who cross the roadway. Raised-curb corner islands and center channelizing or divisional islands can be used as refuge areas. Refuge islands for pedestrians and bicyclists crossing a wide street, for loading or unloading transit riders, or for wheelchair ramps are used primarily in urban areas. **Figure 3 – 11 Pedestrian Refuge Island**, **Figure 3 – 12 Pedestrian Crossing with Refuge Island (Yield Condition)**, and **Figure 3 – 13 Pedestrian Crossing with Refuge Island (Stop Condition)** show divisional islands that support a midblock crosswalk with stop and yield conditions. The distance A shown in the figures is based upon the ***MUTCD***, and shown following the figures.

The location and width of crosswalks, the location and size of transit loading zones, and the provision of curb ramps influence the size and location of refuge islands. Refuge islands should be a minimum of 6 feet wide. Pedestrians and bicyclists should have a clear path through the island and should not be obstructed by poles, sign posts, utility boxes, etc. Sidewalk and shared use path curb ramps in islands shall meet the requirements found in **Section 3.3.10.1.3** of this chapter and **Chapter 8 – Pedestrian Facilities**. Curb ramps that are part of a shared use path shall also meet the requirements of **Chapter 9 – Bicycle Facilities**.

Figure 3-11 Pedestrian Refuge Island

North Main Street, Gainesville, FL

Figure 3-12 Pedestrian Crossing with Refuge Island (Yield Condition)

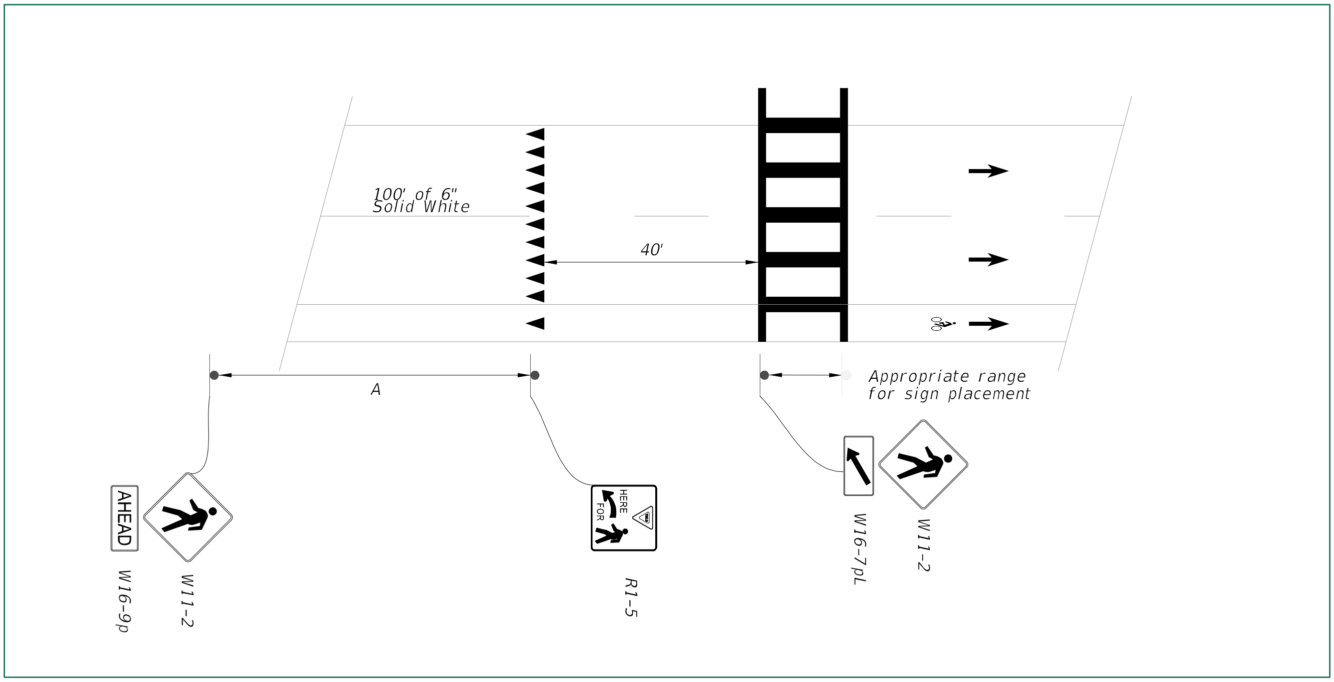
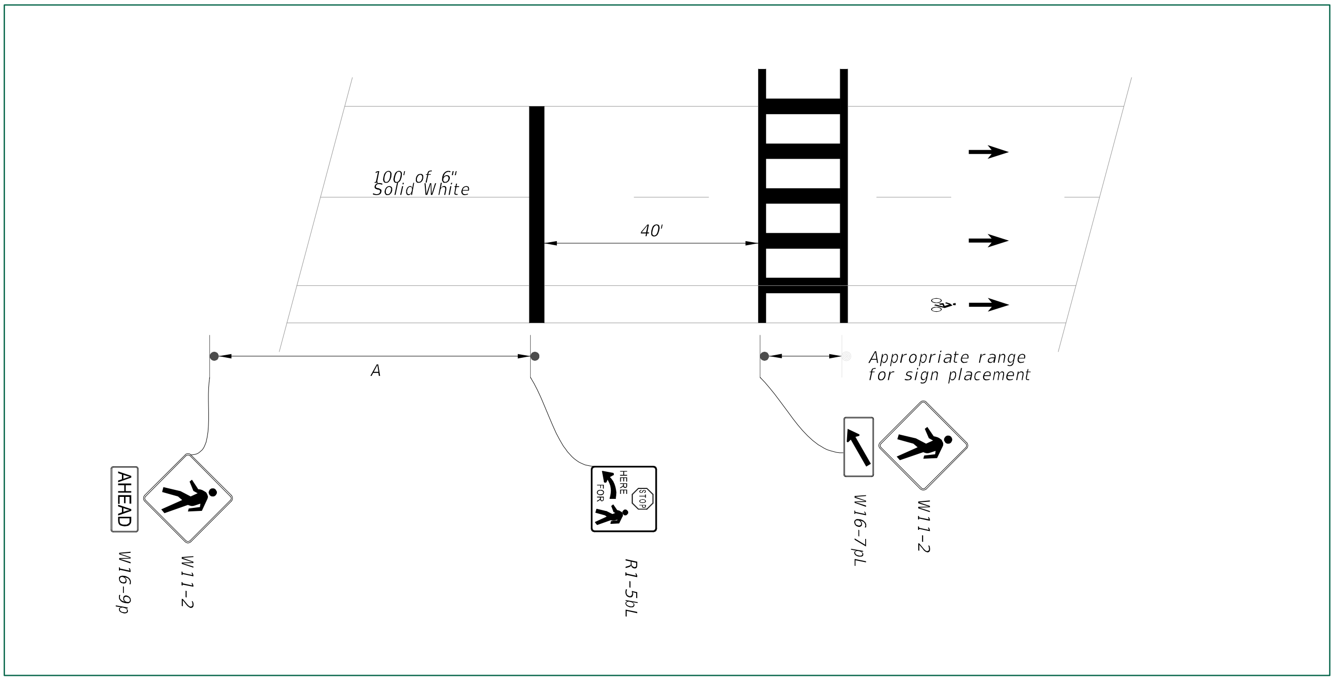


Figure 3-13 Pedestrian Crossing with Refuge Island (Stop Condition)



Note:

1. See following page for distance A.

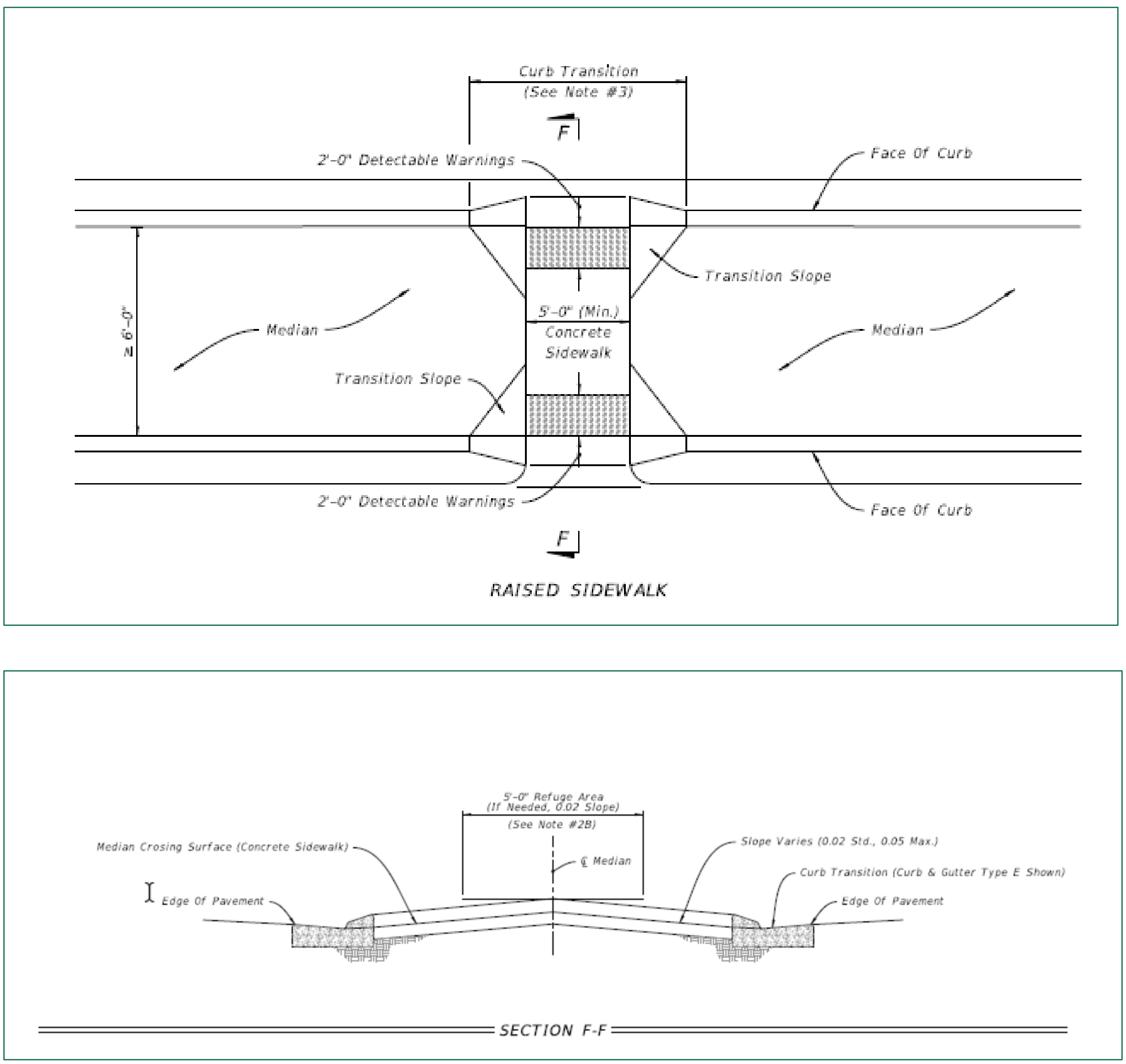
The distance A shown in **Figures 3 – 1~~1~~2** and **3 – 1~~2~~3** for the advance warning sign should be:

Posted Speed (mph)	Advance Placement Distance (feet)
25 or Less	~~100~~155
26 to 35	~~100~~250
36 to 45	~~175~~360

Source: ~~2009~~ MUTCD, ~~with 2012 Revisions,~~ Table 2C-3~~4~~. Guidelines for Advance Placement of Warning Signs. Typical condition is the warning of a potential stop condition.

An example of a pedestrian crossing through a refuge island is shown in **Figure 3 – 14 Pedestrian Crossing in Refuge Island**. Other options are shown in the FDOT's ***Standard Plans 522-002 Detectable Warnings and Sidewalk Curb Ramps.***

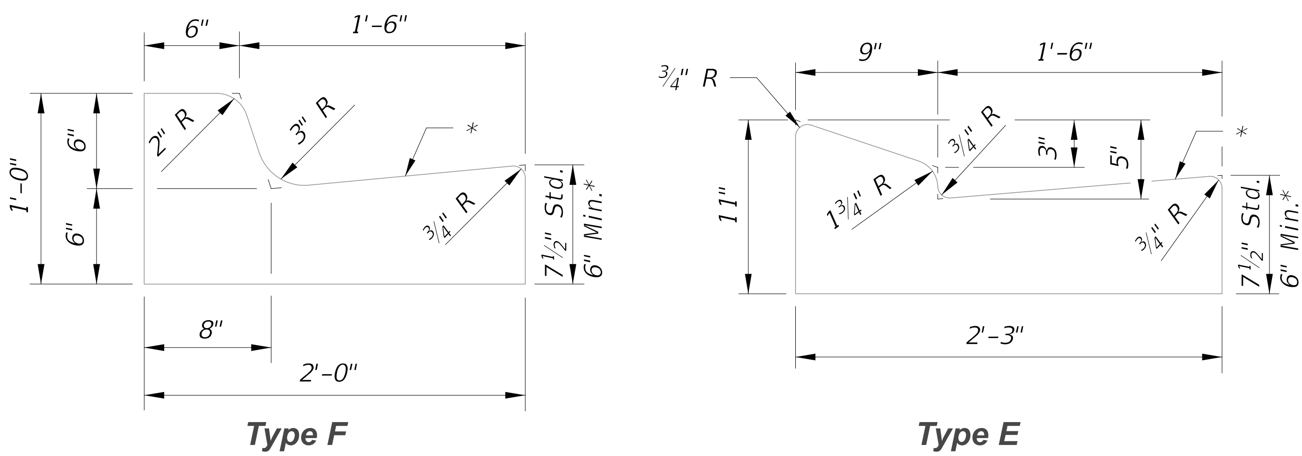
Figure 3-14 Pedestrian Crossing in Refuge Island



3.3.7.7 Curbs

Curbs may be used to provide drainage control and to improve delineation of the roadway. Curbs are generally designed with a gutter to form a combination curb and gutter section. In Florida, the standard curb of this type is 6 inches in height. See **Figure 3 – 15 Standard Detail for FDOT Type F and E Curbs** for examples of sloping curbs. These curbs are not to be used on facilities with design speeds greater than 45 mph. See **Chapter 4 – Roadside Design** for additional design criteria on the use of curbs.

Figure 3-15 Standard Detail for FDOT Type F and E Curbs



3.3.7.8 Parking

Where parking is needed, and adequate off-street parking facilities are not available or feasible, on-street parking may be necessary. On-street parking is allowed on facilities with posted speeds of 35 mph or less. It is typically located at the outside edge of the roadway between the traveled way and the sidewalk. On streets with a posted speed of 25 mph or less, parking may be located within the median in downtown urban centers. On-street parking may be either parallel or angle (traditional or reverse).

On-street parking may help manage traffic speeds, and provides separation between the sidewalk and travel lanes. It may also decrease through capacity, reduce traffic flow, and increase crash potential.

3.3.7.8.1 Parallel Parking Lanes

Minimum parking lane widths for parallel parking are provided in **Table 3 – 25 Minimum Parallel Parking Lane Width**.

If on-street parking is provided adjacent to a bike lane, a buffer zone should be provided to reduce the potential for a car door opening into the bike lane (door zone). The buffer zone between the bike lane and on-street parking should be at least 3’ wide, however 4’ is preferred. See **Figure 9 – 18 Buffered Bicycle Lane Markings with On-Street Parking** for more information.

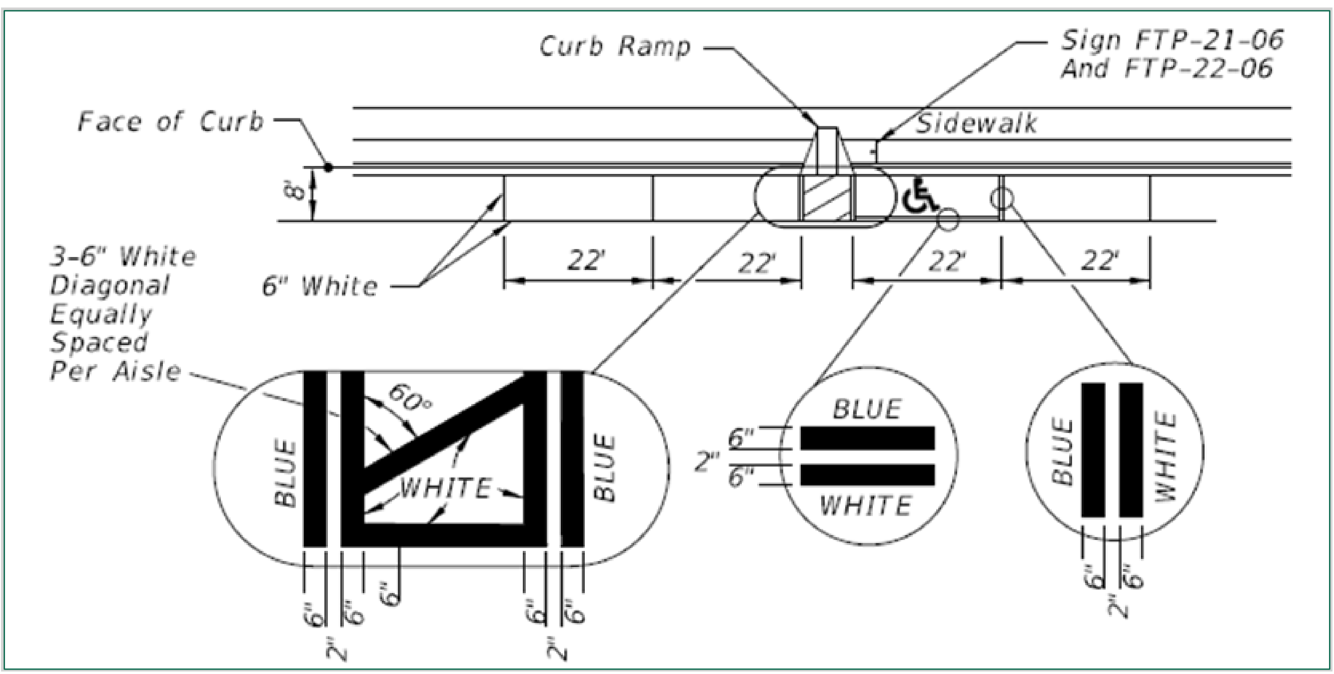
Table 3-25 Minimum Parallel Parking Lane Width

Facility	Posted Speed (mph)	Parallel Parking Lane Width[1] (Feet)
Arterial	≤ 35 mph	8[2]
Collector	≤ 35 mph	8[2,3]
Local	≤ 35 mph	8[2,3]

Notes:
1. Width measured to face of curb.
2. A parking lane width of 10 to 12 feet is desirable where delivery trucks need to be accommodated.
3. May be reduced to 7 feet minimum in residential areas or with posted speeds 25 mph or less, where only passenger vehicles need to be accommodated.

See **Figure 3 – 16** for example details for the signing and marking of parallel parking spaces. The ***MUTCD***~~*MUTCD*~~ provides additional examples of how on-street parking may be marked.

Figure 3-16 Signing and Marking of Parallel Parking Spaces



3.3.7.8.2 Angle Parking

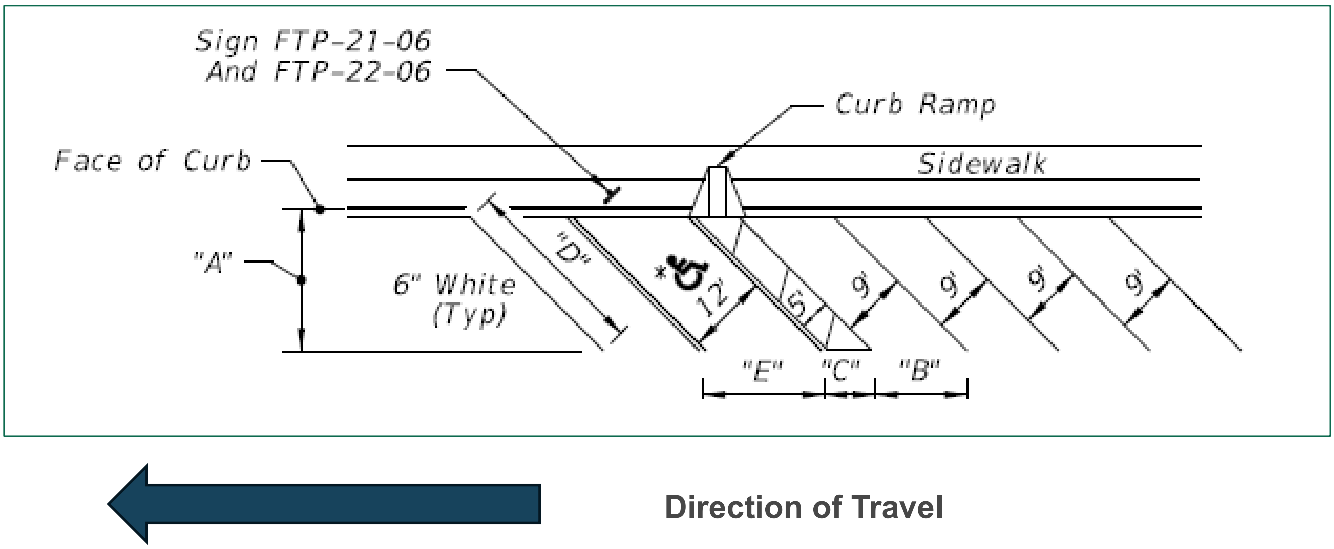
Under certain circumstances, angle parking is an allowable form of street parking. Consideration must be given to the specific function and width of the street, the adjacent land use, traffic volume, and posted speed, as well as existing and anticipated traffic operations.

Angle parking presents special problems because of the varying lengths of vehicles and the sight distance problems associated with vans and recreational vehicles. The extra length of such vehicles may interfere with the traveled way. When reverse angle parking is proposed for on-street parking, a raised median may be used to discourage front in parking and access from the opposite direction of travel.

Angle parking typically requires a minimum of 17 to 18 feet between the curb face or edge of pavement and traveled way.

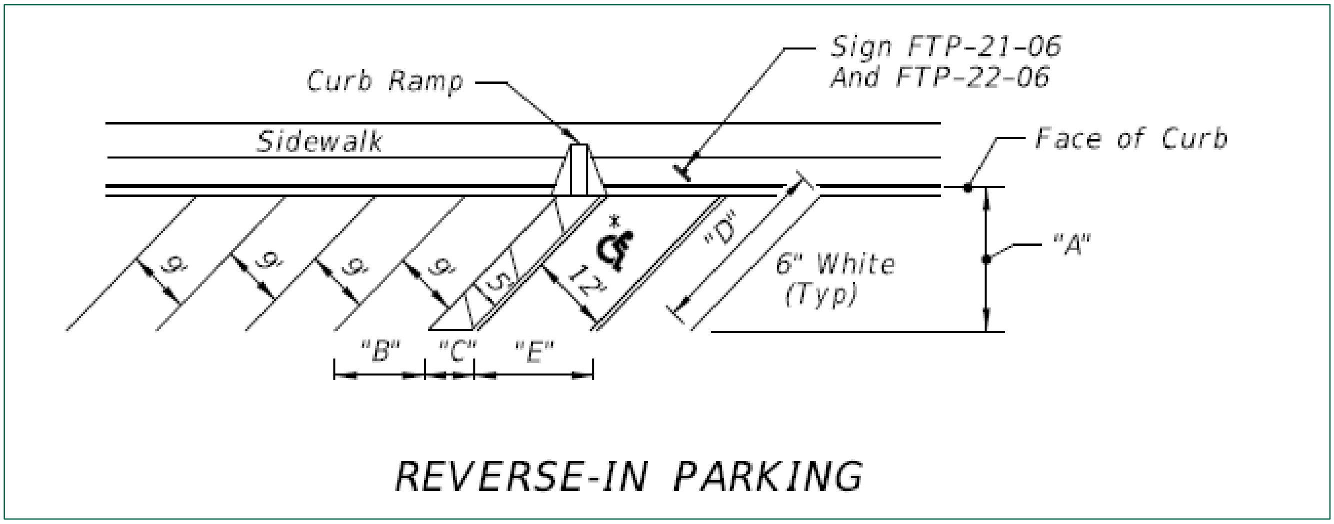
See **Figure 3 – 17 Signing and Marking of 45 degree Forward-In Angle Parking** and **Figure 3 – 18 Signing and Marking of 45 degree Reverse-In Angle Parking** for examples of angle parking.

Figure 3-17 Signing and Marking for 45 degree Forward-In Angle Parking



Dimensions				
“A”	**“B”**	**“C”**	**“D”**	**“E”**
17’ 0”	12’ 9”	7’ 0”	24’ 0”	17’ 0”

Figure 3-18 Signing and Marking for 45 degree Reverse-In Angle Parking



Direction of Travel

Dimensions				
"A"	**"B"**	**"C"**	**"D"**	**"E"**
17' 0"	12' 9"	7' 0"	24' 0"	17' 0"

3.3.7.8.3 Cross Slope

Cross slopes on parking lanes may be 0.015 to 0.05. Portions of parking lanes that are reserved for parking and access isles for people with disabilities are to have cross slopes not exceeding 2%. See **Section 3.3.7.8.4** for further information on accessibility requirements.

The height of the curb, pavement cross slope, utilities, street furniture, and landscaping can all affect the functionality of on-street parking. A bilevel sidewalk can help mitigate the differences in diverse elevations between the roadway, on-street parking, and access to buildings.

3.3.7.8.4 ADA Requirements

In addition to the criteria provided in this section, accessible parking spaces shall be included with on-street parking in accordance with the requirements of the ***2006 Americans with Disabilities Act Standards for Transportation Facilities*** as required by 49 C.F.R 37.41 or 37.43 and the ***2020 Florida Building Code, Accessibility (7th Edition)*** as required by 61G20-4.002. Additionally, the ***U.S. Access Board's (Proposed Public Rights-of-Way***

Accessibility Guidelines, Section R309 On-Street Parking provides the latest direction on accessible design requirements that should be followed.

Figure 3 – 17 Signing and Marking for 45 degree Forward-In Angle Parking and **Figure 3 – 18 Signing and Marking for 45 degree Reverse-In Angle Parking** provide examples of dimensions, signing and marking of on-street parking including accessible parking spaces. The FDOT's ***Standard Plans*** provide further information on the Universal Symbol of Accessibility (Accessible Parking Pavement Marking) and the required signage designating accessible parking spaces.

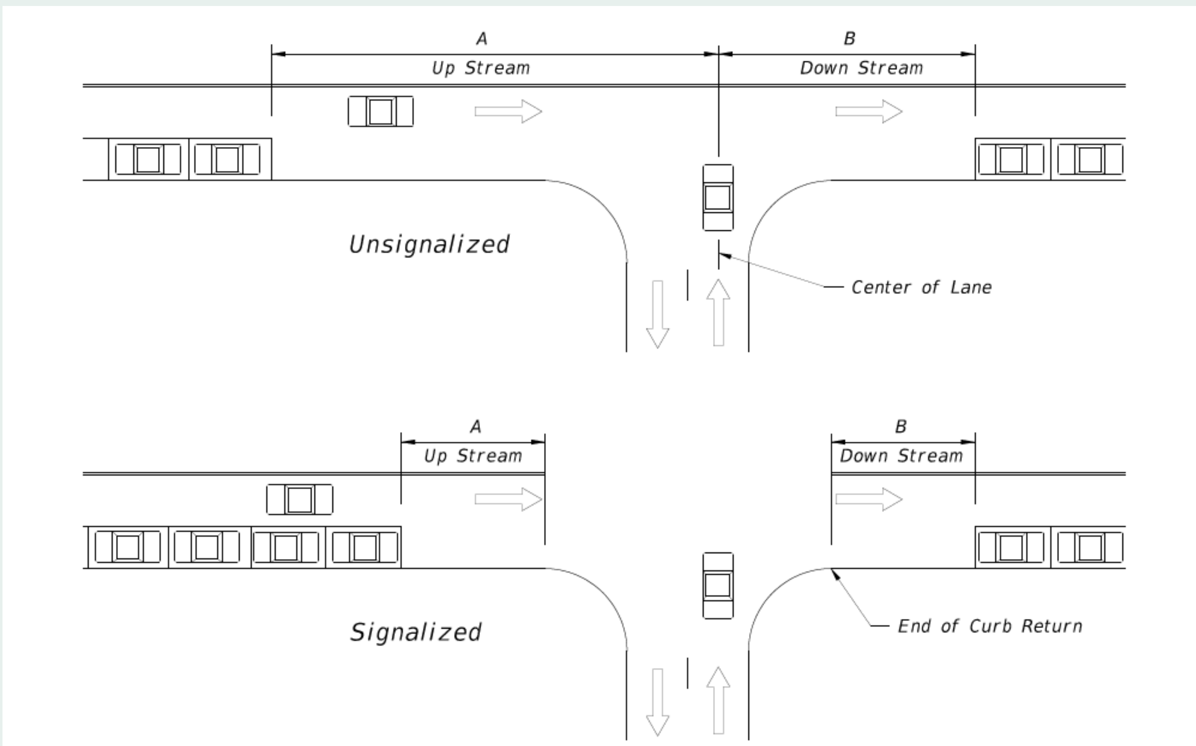
3.3.7.8.5 Parking Restrictions

On-street parking space boundaries shall be established in accordance with the restrictions identified in ***F.S. 316.1945***, which restricts parking near driveways, intersections, crosswalks, railroad crossings, fire hydrants and fire stations.

On-street parking shall be located no closer to driveways and intersections than the distances provided in **Table 3 – 26 Parking Restrictions for Driveways, Intersections, and Mid-Block Crosswalks**. This includes mid-block crossings and roundabout approaches. Midblock crossings on streets with parking should include curb extensions or bulb-outs to improve a driver's and pedestrian's ability to see each other. See **Chapter 15 – Traffic Calming** for more information.

Table 3-26 Parking Restrictions for Driveways, Intersections and Mid-Block Crosswalks

Control Type	Posted Speed (mph)	A Up Stream (feet)	B Down Stream (feet)	
			2-Lane	4-Lane or More
Unsignalized	< 35	90	60	45
	35	105	70	50
Signalized and 4-Way Stop Controlled	< 35	30	30	30
	35	50	50	50



Notes:
1. For entrances to one-way streets, the downstream restriction (B) may be reduced to 20 feet.
2. Do not place parking within 20 feet of the nearest edge of a marked crosswalk.

3.3.7.8.6 Signing and Marking

Signing and marking of on-street parking shall conform to *MUTCD*~~*MUTCD*~~ as well as ADA requirements identified in **Section 3.3.7.8.4**.

3.3.7.9 Right of Way

The acquisition of sufficient right of way is necessary in order to provide space for a safe street or highway. The width of the right of way required depends on the design of the roadway, the arrangement of bridges, underpasses and other structures, and the need for cuts or fills. The right of way acquired should be sufficient to:

- Allow development of the full cross section, including adequate medians and roadside clear zones. Determination of the necessary width requires that adequate consideration also be given to the accommodation of utility poles beyond the clear zone.
- Allow the layout of safe intersections, interchanges, and other access points.
- Allow adequate sight distance at all points, particularly on horizontal curves, at an intersection, and other access points.
- Allow, where appropriate, additional buffer zones to improve roadside safety, noise attenuation, and the overall aesthetics of the street or highway.
- Allow adequate space for placement of pedestrian and bicycle facilities, including curb ramps, bus bays, and transit shelters, where applicable.
- Allow for future lane additions, increases in cross section, or other improvement. Frontage roads should also be considered in the ultimate development of many high volume facilities.
- Allow treatment of stormwater runoff.
- Allow for construction of future intersection improvements, such as turn lanes, bicycle and pedestrian facilities or over and underpasses.
- Allow corner cuts for upstream corner crossing drainage systems and placement of poles, boxes, and other visual screens out of the critical sight triangle.
- Allow landscaping and irrigation as required for the project.

The acquisition of wide rights of way is costly, but it may be necessary to allow the construction and future improvement of safe streets and highways. The minimum right of way should be at least 50 feet for all two-lane roads. For pre-existing conditions, when the existing right of way is less than 50 feet, efforts should be made to acquire the necessary right of way.

Local cul-de-sac and dead end streets having an ADT of less than or equal to 400 and a length of 600 feet or less, may utilize a right of way of less than 50 feet, if all elements of the typical section meet the standards included in this Manual.

The right of way for frontage roads may be reduced depending on the typical section requirements and the ability to share right of way with the adjacent street or highway facility.

3.3.7.10 Changes in Typical Section

3.3.7.10.1 General Criteria

Changes in cross section should be avoided. When changes in widths, slopes, or other elements are necessary, they should be affected in a smooth, gradual fashion.

3.3.7.10.2 Lane Deletions and Additions

The addition or deletion of traffic or bicycle lanes should be undertaken on tangent sections of roadways. The approach to lane deletions and additions should have ample advance warning and sight distance.

The termination of lanes (including auxiliary lanes) shall meet the general requirements for merging lanes. See **Section 3.3.9.3.1** for additional information.

Where additional lanes are intermittently provided on two-lane, two-way highways, median separation should be considered.

3.3.7.10.3 Preferential Lanes

To increase the efficiency and separation of different vehicle movements, preferential use lanes, such as bike lanes and bus lanes, should be considered. These lanes are often an enhancement to corridor safety and increase the horizontal clearance to roadside aboveground fixed objects. The ~~***MUTCD, Chapter 3D***~~***MUTCD, Chapter 3E*** provides further information on preferential lane markings. See **Chapter 9 – Bicycle Facilities** for information on marking bicycle lanes.

3.3.7.10.4 Structures

The pavement, median, and shoulder width, and sidewalks should be carried across structures such as bridges and box culverts. Shoulder widths for multi-lane rural divided highway bridges may be reduced as shown in **Table 3 – 22 Minimum Shoulder Widths for Flush Shoulder Rural Highways**. The designer should evaluate the economic practicality of utilizing dual versus single bridges for roadway sections incorporating wide medians.

The minimum roadway width for bridges on urban streets with curb and gutter shall be the same as the curb-to-curb width of the approach roadway. Sidewalks on the approaches should be carried across all structures. Curbed sidewalks should not be used adjacent to traffic lanes when design speeds exceed 45 mph. When the bridge rail (barrier wall) is placed between the traffic and sidewalk, it should be offset a minimum distance of 2½ feet from the edge of the travel lane, wide curb lane or bicycle lane. For long (500 feet or greater), and/or high level bridges, it is desirable to provide an offset distance that will accommodate a disabled vehicle. The transition from the bridge to the adjacent roadway section may be made by dropping the curb at the first intersection or well in advance of the traffic barrier, or reducing the curb in front of the barrier to a low sloping curb with a gently sloped traffic face. See **Chapter 17 – Bridges and Other Structures** for additional requirements.

- Lateral Offset - Supports for bridges, barriers, or other structures should be placed at or beyond the required shoulder. Where possible, these structures should be located outside of the required clear zone. See **Chapter 4 – Roadside Design** for additional information on lateral offsets for structures.
- Vertical Clearance - Vertical clearance should be adequate for the type of expected traffic. Freeways and arterials shall have a vertical clearance of at least 16 feet-6 inches (includes 6 inch allowance for future resurfacing). Other streets and highways should have a clearance of 16 feet unless the provision of a reduced clearance is fully justified by a specific analysis of the situation (14 feet minimum). The minimum vertical clearance for a pedestrian or shared use bridge over a roadway is 17 feet. The minimum vertical clearance for a bridge over a railroad is 23 feet; however additional clearance may be required by the rail owner.
- End Treatment - The termini of guardrails, bridge railings, abutments, and other structures should be constructed to protect vehicles and their occupants from serious impact. Requirements for end treatment of structures are given in **Chapter 4 - Roadside Design**.

3.3.8 Access Control

All new facilities (and existing when possible) should have some degree of access control since each point of access produces a traffic conflict. The control of access is one of the most effective, efficient, and economical methods for improving the capacity and safety characteristics of streets and highways. The reduction of the frequency of access points and the restriction of turning and crossing maneuvers, which should be primary objectives, is accomplished more effectively by the design of the roadway geometry than by the use of traffic control devices. Design criteria for access points are presented under the general requirements for intersection design.

Additional information on access management can be found in ***Rule Chapter 14-97 State Highway System Access Control Classification System, Florida Administrative Code***. The FDOT's ~~***Access Management Guidebook (2019)***~~***Access Management Guidebook (2023)*** provides further information on designing roadways and connections to support access management .

3.3.8.1 Justification

The justification for control of access should be based on several factors, including safety, capacity, economics, and aesthetics.

3.3.8.2 General Criteria

3.3.8.2.1 Location of Access Points

All access locations should have adequate sight distance available for the safe execution of entrance, exit, and crossing maneuvers.

Locations of access points near structures, decision points, or the termination of street or highway lighting should be avoided.

Driveways should not be placed within the influence zone of intersections or other points that would tend to produce traffic conflict.

3.3.8.2.2 Spacing of Access Points

The spacing of access points should be adequate to prevent conflict or mutual interference of traffic flow.

Separation of entrance and exit ramps should be sufficient to provide adequate distance for required weaving maneuvers.

Adequate spacing between access and decision points is necessary to avoid burdening the driver with the need for rapid decisions or maneuvers.

Frequent median openings should be avoided.

The use of a frontage road or other auxiliary roadways is recommended on arterials and higher classifications where the need for direct driveway or minor road access is frequent.

3.3.8.2.3 Restrictions of Maneuvers

Where feasible, the number and type of permitted maneuvers (crossing, turning slowing, etc.) should be restricted.

The restriction of crossing maneuvers may be accomplished by the use of grade separations and continuous raised medians.

The restriction of left turns is achieved most effectively by continuous medians.

Channelization should be considered for the purposes of guiding traffic flow and reducing vehicle conflicts.

3.3.8.2.4 Auxiliary Lanes

Deceleration lanes for right turn exits (and left turns, where permitted) should be provided on all high-speed facilities. These turn lanes should not be excessive or continuous since they complicate pedestrian crossings and bicycle/ motor vehicle movements.

Storage (or deceleration lanes) to protect turning vehicles should be provided, particularly where turning volumes are significant.

Special consideration should be given to the provisions for deceleration, acceleration, and storage lanes in commercial or industrial areas with significant truck/bus traffic.

3.3.8.2.5 Grade Separation

Grade separation interchange design should be considered for junctions of high-volume arterial streets and highways.

Grade separation (or an interchange) should be utilized when the expected traffic volume exceeds the intersection capacity.

Grade separation should be considered to eliminate conflict or long waiting periods at potentially hazardous intersections.

3.3.8.2.6 Roundabouts

Roundabouts have proven safety and operational characteristics and should be evaluated as an alternative to conventional intersections whenever practical. Modern roundabouts, when correctly designed, are a proven safety countermeasure to conventional intersections, both stop controlled and signalized. In addition, when constructed in appropriate locations, drivers will experience less delay with modern roundabouts. ~~***NCHRP Report 672 Roundabouts: An Informational Guide***~~***NCHRP Report 1043: Guide for Roundabouts***, ~~is adopted by FHWA and~~ establishes criteria and procedures for the justification, operational and safety analysis of modern roundabouts in the United States. The modern roundabout is characterized by the following:

- A central island of sufficient diameter to accommodate vehicle tracking and to provide sufficient deflection to promote lower speeds
- Entry is by gap acceptance through a yield condition at all legs

- Speeds through the intersection of 20 - 25 mph.
- Single or multilane configurations.

Roundabouts should be considered under the following conditions:

1. New construction
2. Reconstruction
3. Traffic Operations improvements
4. Resurfacing (3R) with Right of Way acquisition
5. Need to reduce frequency and severity of crashes

3.3.8.3 Control for All Limited Access Highways

Entrances and exits on the right side only are highly desirable for all limited access highways. Acceleration and deceleration lanes are mandatory. Intersections shall be accomplished by grade separation (interchange) and should be restricted to connect with arterials or collector roads.

The control of access on freeways should conform to the requirements given in **Table 3 – 27 Access Control for All Limited Access Highways**. The spacing of exits and entrances should be increased wherever possible to reduce conflicts. Safety and capacity characteristics are improved by restricting the number and increasing the spacing of access points.

Table 3-27 Access Control for All Limited Access Highways

Minimum Spacing		
	Urban	**Rural**
Interchanges	1 to 3 miles	3 to 25 miles
Maneuver Restrictions		
Crossing Maneuvers	Via Grade Separation Only	
Exit and Entrance	From Right Side Only	
Turn Lane Required	Acceleration Lane at all Entrances Deceleration Lane at all Exits	

3.3.8.4 Control of Urban and Rural Streets and Highways

The design and construction of urban, as well as rural, highways should be governed by the general criteria for access control previously outlined. In addition, the design of urban streets should be in accordance with the criteria listed below:

- The general layout of local and collector streets should follow a branching network, rather than a highly interconnected grid pattern.
- The street network should be designed to reduce, consistent with origin/destination requirements, the number of crossing and left turn maneuvers.
- The design of the street layout should be predicated upon reducing the need for traffic signals.
- The use of a public street or highway as an integral part of the internal circulation pattern for commercial property should be discouraged.
- The number of driveway access points should be restricted as much as possible through areas of strip development.
- Special consideration should be given to providing turn lanes (auxiliary lane for turning maneuvers) where the total volume or truck/bus volume is high.
- Major traffic generators may be exempt from the restrictions on driveway access if the access point is designed as a normal intersection adequate to handle the expected traffic volume.

These are minimum requirements only; it is generally desirable to use more stringent criteria for control of access.

The design of rural highways should be in accordance with the general criteria for access control for urban streets. The use of acceleration and deceleration lanes on all high-speed highways, particularly if truck and bus traffic is significant, is strongly recommended.

3.3.8.5 Land Development

It should be the policy of each agency with responsibility for street and highway design, construction, or maintenance to promote close liaison with utility, lawmaking, zoning, building, and planning agencies. Cooperation should be solicited in the formulation of laws, regulations, and master plans for land use, zoning, and road construction. Further requirements and criteria for access control and land use relationships are given in **Chapter 1 – Planning and Land Development**.

3.3.9 Intersection Design

Intersections increase traffic conflicts and the demands on the driver, and are inherently hazardous locations. The design of an intersection should be predicated on reducing motor vehicle, bicycle, and pedestrian conflicts, minimizing the confusion and demands on the driver for rapid and/or complex decisions, and providing for smooth traffic flow. The location and spacing of intersections should follow the requirements presented in Section C.8 Access Control, this chapter. Intersections should be designed to minimize time and distance of all who pass through or turn at an intersection.

The additional effort and expense required to provide a high quality intersection is justified by the corresponding safety benefits. The overall reduction in crash potential derived from a given expenditure for intersection improvements is generally much greater than the same expenditure for improvements along an open roadway. Properly designed intersections increase capacity, reduce delays, and improve safety.

One of the most common deficiencies that may be easy to correct is lack of adequate left turn storage.

The requirements and design criteria contained in this section are applicable to all driveways, intersections, and interchanges. All entrances to, exits from, or interconnections between streets and highways are subject to these design standards.

3.3.9.1 General Criteria

The layout of a given intersection may be influenced by constraints unique to a particular location or situation. The design shall conform to sound principles and criteria for safe intersections. The general criteria include the following:

- The layout of the intersection should be as simple as is practicable. Complex intersections, which tend to confuse and distract the driver, produce inefficient and hazardous operations.
- The intersection arrangement should not require the driver to make rapid or complex decisions.
- The layout of the intersection should be clear and understandable so a proliferation of signs, signals, or markings is not required to adequately inform and direct the driver.
- The design of intersections, particularly along a given street or highway, should be as consistent as possible.
- The approach roadways should be free from steep grades and sharp horizontal or vertical curves.
- Intersections with driveways or other roadways should be as close to right angle as possible.
- Adequate sight distance should be provided to present the driver a clear view of the intersection and to allow for safe execution of crossing and turning maneuvers.
- The design of all intersection elements should be consistent with the design speeds of the approach roadways.
- The intersection layout and channelization should encourage smooth flow and discourage wrong way movements.
- Special attention should be directed toward the provision of safe roadside clear zones.
- The provision of auxiliary lanes should be in conformance with the criteria set forth in **Section 3.3.8 Access Control**, this chapter.
- The requirements for bicycle and pedestrian movements should receive special consideration.

3.3.9.2 Sight Distance

Inadequate sight distance is a contributing factor in the cause of a large percentage of intersection crashes. The provision of adequate sight distance at intersections is absolutely essential and should receive a high priority in the design process.

3.3.9.2.1 General Criteria

General criteria to be followed in the provision of sight distance include the following:

- Sight distance exceeding the minimum stopping sight distance should be provided on the approach to all intersections (entrances, exits, stop signs, traffic signals, and intersecting roadways). The use of proper approach geometry free from sharp horizontal and vertical curvature will normally allow for adequate sight distance.
- The approaches to exits or intersections (including turn, storage, and deceleration lanes) should have adequate sight distance for the design speed and also to accommodate any allowed lane change maneuvers.
- Adequate sight distance should be provided on the through roadway approach to entrances (from acceleration or merge lanes, stop or yield signs, driveways, or traffic signals) to provide capabilities for defensive driving. This lateral sight distance should include as much length of the entering lane or intersecting roadway as is feasible. A clear view of entering vehicles is necessary to allow through traffic to aid merging maneuvers and to avoid vehicles that have "run" or appear to have the intention of running stop signs or traffic signals.
- Approaches to school or pedestrian crossings and crosswalks should have sight distances exceeding the minimum values. This should also include a clear view of the adjacent pedestrian pathways or shared use paths.
- Sight distance in both directions should be provided for all entering roadways (intersecting roadways and driveways) to allow entering vehicles to avoid through traffic. See **Section 3.3.9.2.4** for further information.
- Safe stopping sight distances shall be provided throughout all intersections, including turn lanes, speed change lanes, and turning roadways.
- The use of lighting (**Chapter 6 – Lighting**) should be considered to improve intersection sight distance for night driving.

3.3.9.2.2 Obstructions to Sight Distance

The provisions for sight distance are limited by the street or highway geometry and the nature and development of the area adjacent to the roadway. Where line of sight is limited by vertical curvature or obstructions, stopping sight distance shall be based on the eye height of 3.50 feet and an object height of 2.0 feet. At exits or other locations where the driver may be uncertain as to the roadway alignment, a clear view of the pavement surface should be provided. At locations requiring a clear view of other vehicles or pedestrians for the safe execution of crossing or entrance maneuvers, the sight distance should be based on a driver's eye height of

3.50 feet and an object height of 3.00 feet (preferably 1.50 feet). The height of eye for truck traffic may be increased for determination of line of sight obstructions for intersection maneuvers. Obstructions to sight distance at intersections include the following:

- Any property not under the highway agency's jurisdiction, through direct ownership or other regulations, should be considered as an area of potential sight distance obstruction. Based on the degree of obstruction, the property should be considered for acquisition by deed or easement.
- Areas which contain vegetation (trees, shrubbery, grass, etc.) that cannot easily be trimmed or removed by regular maintenance activity should be considered as sight obstructions.
- Parking lanes shall be considered as obstructions to line of sight. Parking shall be prohibited within clear areas required for sight distance at intersections.
- Large (or numerous) poles or support structures for lighting, signs, signals, or other purposes that significantly reduce the field of vision within the limits of clear sight shown in **Figure 3 – 20 Departure Sight Triangle** in **Section 3.3.9.2.4** may constitute sight obstructions. Potential sight obstructions created by poles, supports, and signs near intersections should be carefully investigated.

In order to ensure the provision for adequate intersection sight distance, on-site inspections should be conducted before and after construction, including placement of signs, lighting, guardrails, or other objects and how they impact intersection sight distance.

3.3.9.2.3 Stopping Sight Distance

The provision for safe stopping sight distance at intersections and on turning roadways is even more critical than on open roadways. Vehicles are more likely to be traveling in excess of the design or posted speed and drivers are frequently distracted from maintaining a continuous view of the upcoming roadway.

3.3.9.2.3.1 Approach to Stops

The approach to stop signs, yield signs, or traffic signals should be provided with a sight distance no less than values given in **Table 3 – 28 Minimum Stopping Sight Distance (Rounded Values)**. These values are applicable for any street, highway, or turning roadway. The driver should, at this required distance, have a clear view of the intersecting roadway, as well as the sign or traffic signal.

Where the approach roadway is on a grade or vertical curve, the sight distance should be no less than the values shown in **Figure 3 – 19 Sight Distances for Approach to Stop on Grades**. In any situation where it is feasible, sight distances exceeding those should be provided. This is desirable to allow for more gradual stopping maneuvers and to reduce the likelihood of vehicles running through stop signs or signals. Advance warnings for stop signs are desirable.

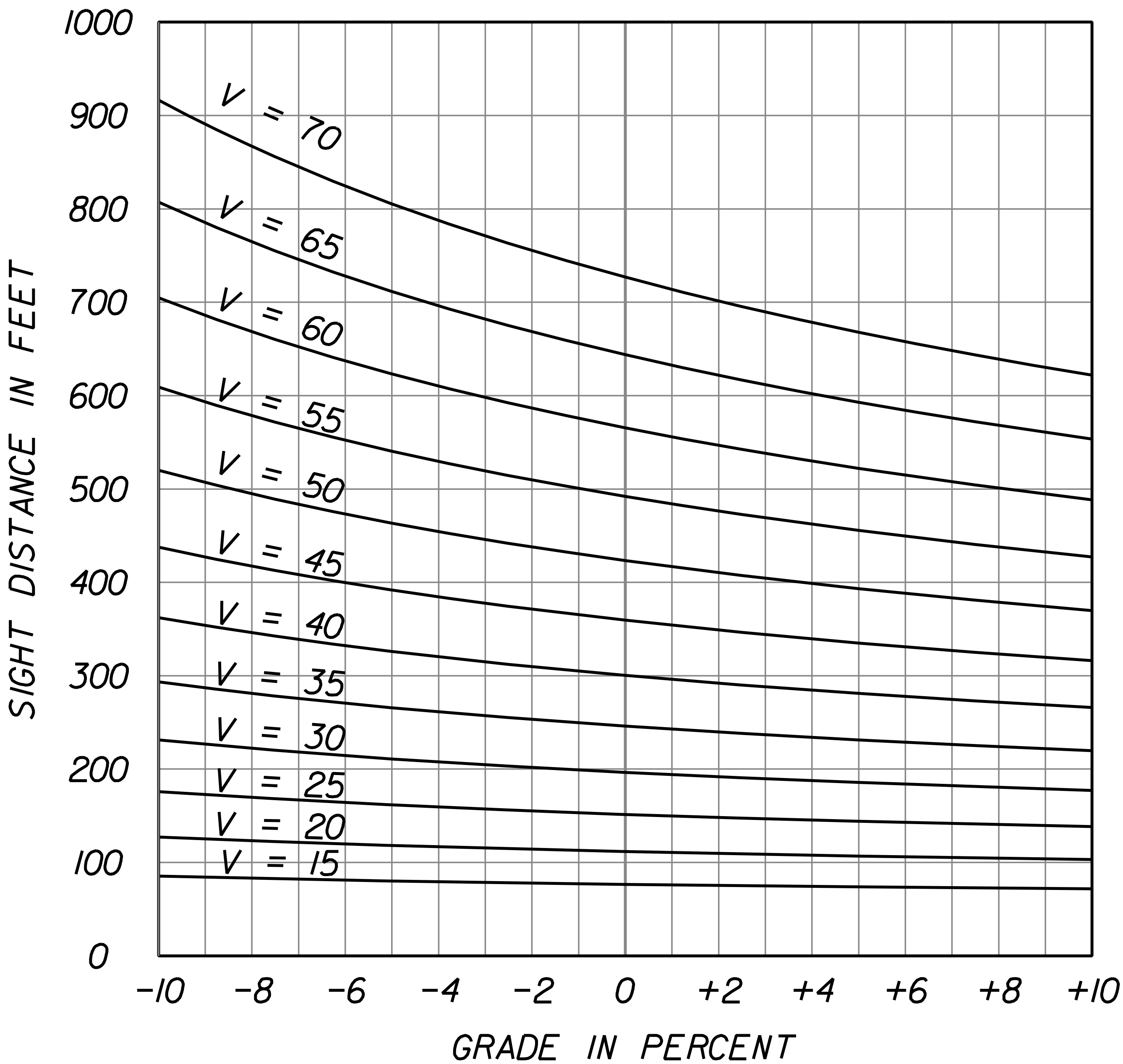
Table 3-28 Minimum Stopping Sight Distance (Rounded Values)

Design Speed (mph)	15	20	25	30	35	40	45	50	55	60	65	70
Stopping Sight Distance (feet)	80	115	155	200	250	305	360	425	495	570	645	730

3.3.9.2.3.2 On Turning Roads

The required stopping sight distance at any location on a turning roadway (loop, exit, etc.) shall be based on the design speed at that point. Ample sight distance should be provided since the driver is burdened with negotiating a curved travel path and the available friction factor for stopping has been reduced by the roadway curvature. The minimum sight distance values are given in **Table 3 – 28 Minimum Stopping Sight Distance (Rounded Values)** or **Figure 3 – 19 Sight Distances for Approach to Stop on Grades**. Due to the inability of vehicle headlights to adequately illuminate a sharply curved travel path, roadway lighting should be considered for turning roadways.

Figure 3-19 Sight Distances for Approach to Stop on Grades



$$S = 3.675V + \frac{V^2}{30(0.3478 \pm G)}$$

S = Sight Distance

V = Design Speed

G = Grade

3.3.9.2.4 Sight Distance for Intersection Maneuvers

Sight distance is also provided at intersections to allow the drivers of stopped vehicles a sufficient view of the intersecting street or highway to decide when to enter or cross the intersecting street or highway. Sight triangles, which are specified areas along intersection approach legs and across their included corners, shall, where practical, be clear of obstructions that would prohibit a driver's view of potentially conflicting vehicles. Departure sight triangles shall be provided in each quadrant of each intersection approach controlled by stop signs.

Figures 3 – 20~~19~~ Departure Sight Triangle (Traffic Approaching from Left or Right) and **3 – ~~20~~ 21 Intersection Sight Distance** show typical departure sight triangles to the left and to the right of the location of a stopped vehicle on a minor road (stop controlled) and the intersection sight distances for the various movements.

Distance "a" is the length of leg of the sight triangle along the minor road. This distance is measured from the driver's eye in the stopped vehicle to the center of the nearest lane on the major road (through road) for vehicles approaching from the left, and to the center of the nearest lane for vehicles approaching from the right.

Distance "b" is the length of the leg of the sight triangle along the major road measured from the center of the minor road entrance lane. This distance is a function of the design speed and the time gap in major road traffic needed for minor road drivers turning onto or crossing the major road. This distance is calculated as follows:

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

Where:

ISD=Intersection Sight Distance (ft) – length of leg of sight triangle along the major road.

V_{major}= Design Speed (mph) of the Major Road

t_g= Time gap (sec) for minor road vehicle to enter the major road.

Time gap values, t_g, to be used in determination of ISD are based on studies and observations of the time gaps in major road traffic actually accepted by drivers turning onto or across the major road. Design time gaps will vary and depend on the design vehicle, the type of the maneuver, the crossing distance involved in the maneuver, and the minor road approach grade.

For intersections with stop control on the minor road, there are three maneuvers or cases that must be considered. ISD is calculated for each maneuver case that may occur at the intersection. The case requiring the greatest ISD will control. Cases that must be considered are as follows (Case numbers correspond to cases identified in the AASHTO – "A Policy on Geometric Design of Highways and Streets" - ~~2011~~ 2018):

- Case B1 – Left Turns from the Minor (stop controlled) Road
- Case B2 – Right Turns from the Minor (stop controlled) Road
- Case B3 – Crossing the Major Road from the Minor (stop controlled) Road

See **Sections 3.3.9.2.4.3** and **3.3.9.2.4.4** for design time gaps for Case B.

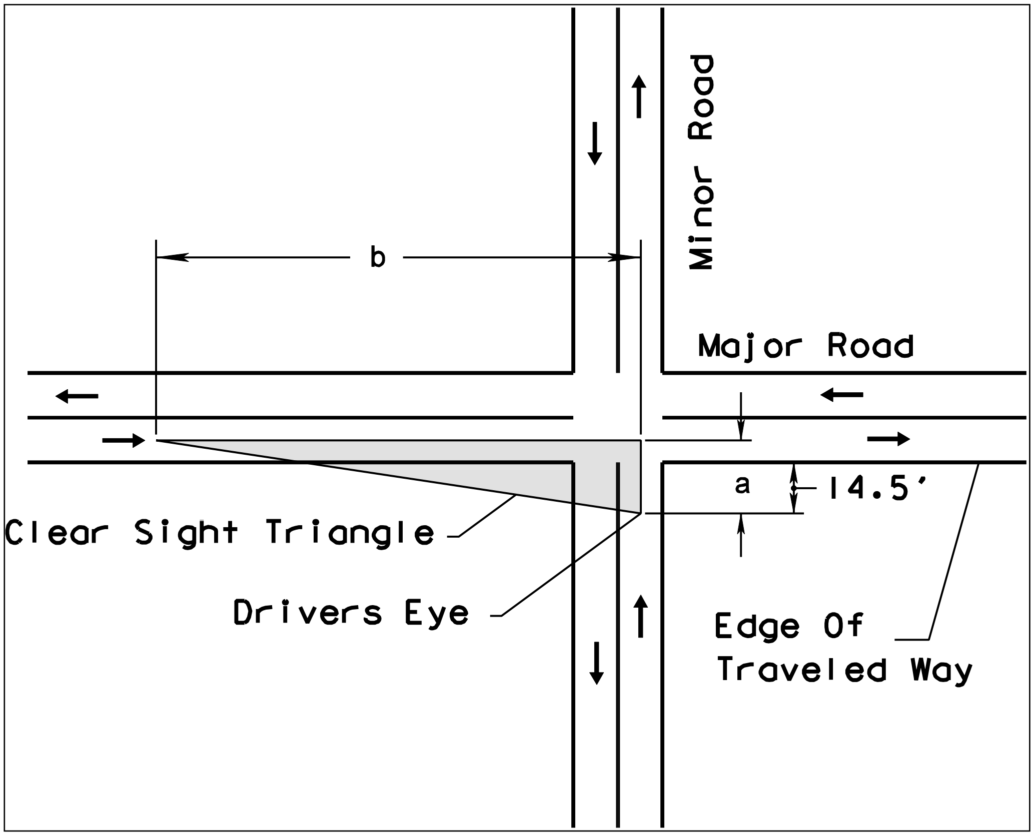
For Intersections with Traffic Signal Control see **Section 3.3.9.2.4.5** (AASHTO Case D).

For intersections with all way stop control see **Section 3.3.9.2.4.6** (AASHTO Case E).

For left turns from the major road see **Section 3.3.9.2.4.7** (AASHTO Case F).

For Roundabouts see **Section 3.3.9.2.4.8** (AASHTO Case G).

Figure 3-20 Departure Sight Triangle (Traffic Approaching from Left or Right)



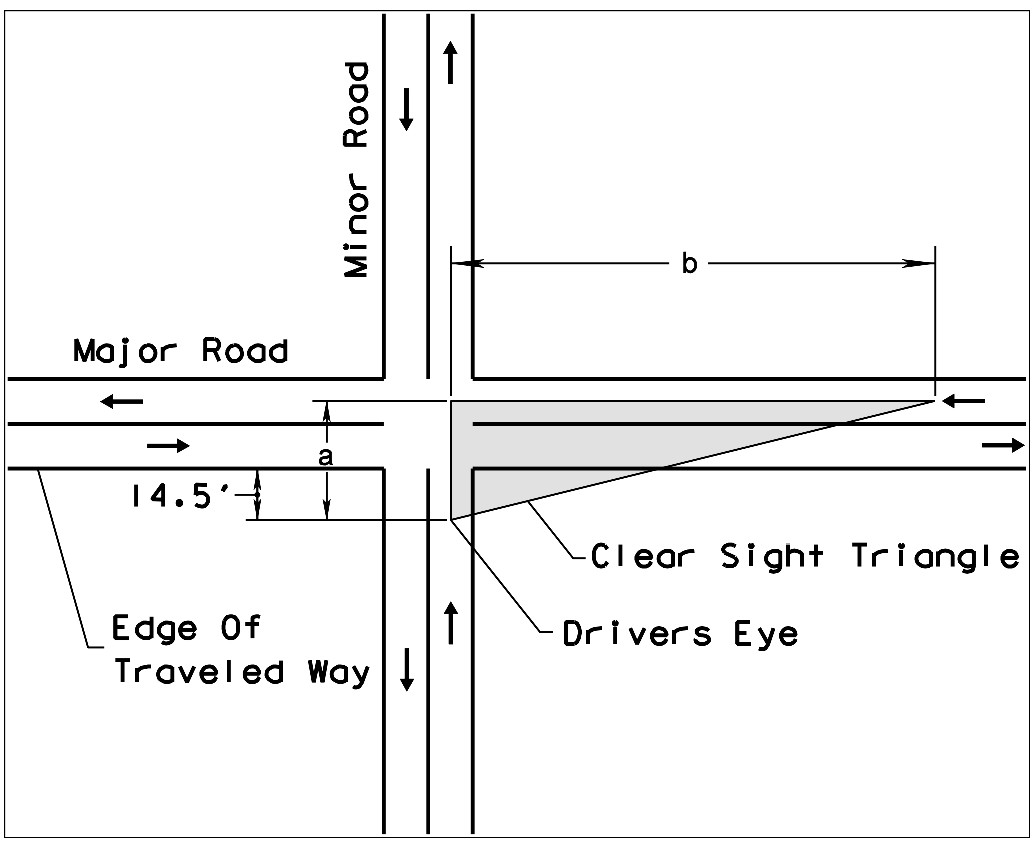
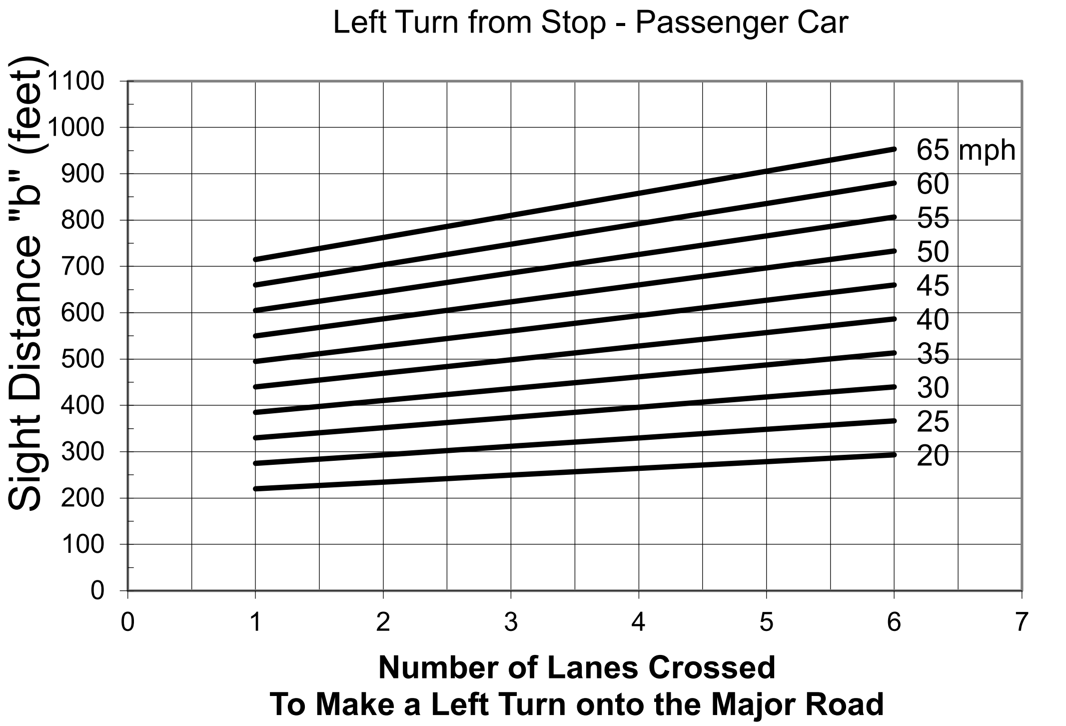
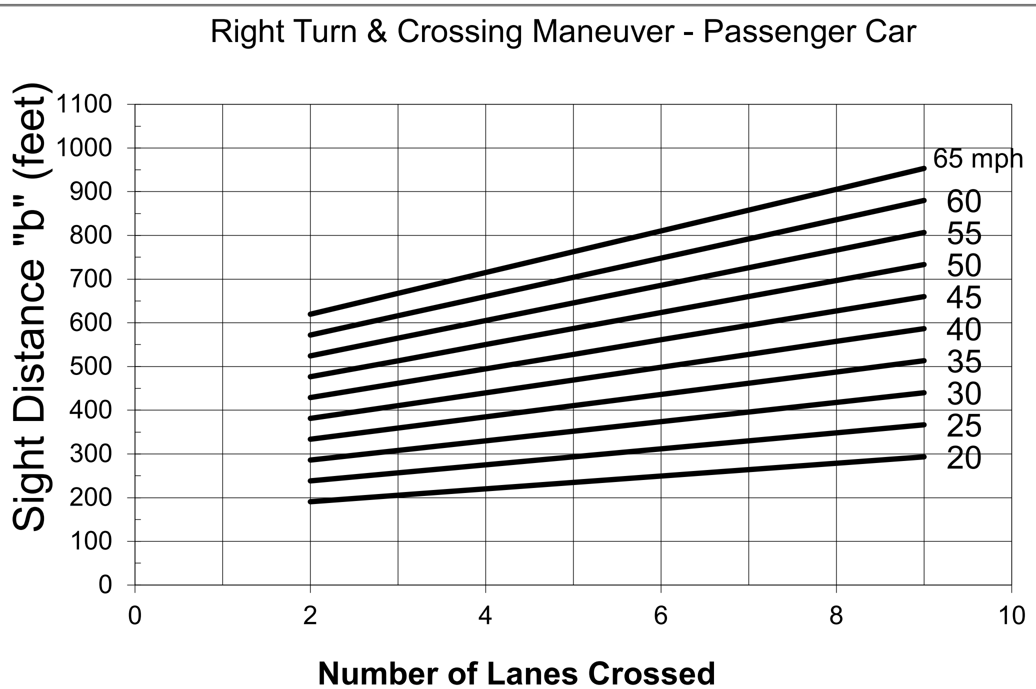


Figure 3-21 Intersection Sight Distance



A vehicle crosses one lane to make a left turn onto a two lane-two way road; crosses two lanes on a four lane undivided road; and so forth. For divided highways, convert median widths to equivalent number of lanes. For medians 25' and greater a two step maneuver may be considered - see text.



For right turn maneuver use distance based on 2 lanes crossed.
For crossing divided highways, convert median widths to equivalent number of lanes. For medians 25' and greater a two step maneuver may be considered - see text.

3.3.9.2.4.1 Driver's Eye Position and Vehicle Stopping Position

The vertex (decision point or driver's eye position) of the departure sight triangle on the minor road shall be a minimum of 14.5 feet from the edge of the major road traveled way. This is based on observed measurements of vehicle stopping position and the distance from the front of the vehicle to the driver's eye. Field observations of vehicle stopping positions found that, where necessary, drivers will stop with the front of their vehicle 6.5 feet or less from the edge of the major road traveled way. Measurements of passenger cars indicate that the distance from the front of the vehicle to driver's eye for the current U.S. passenger car fleet is almost always 8 feet or less.

When executing a crossing or turning maneuver after stopping at a stop sign, stop bar, or crosswalk as required in ***Section 316.123, Florida Statutes***, it is assumed that the vehicle will move slowly forward to obtain sight distance (without intruding into the crossing travel lane) stopping a second time as necessary.

3.3.9.2.4.2 Design Vehicle

Dimensions of clear sight triangles are provided for passenger cars, single unit trucks, and combination trucks stopped on the minor road. It can usually be assumed that the minor road vehicle is a passenger car. However, where substantial volumes of heavy vehicles enter the major road, such as from a ramp terminal, the use of tabulated values for single unit or combination trucks should be considered.

3.3.9.2.4.3 Case B1 - Left Turns from the Minor Road

Design time gap values for left turns from the minor road onto two lane two way major highway are as follows:

Design Vehicle	Time Gap (t_g) in Seconds
Passenger Car	7.5
Single Unit Truck	9.5
Combination Truck	11.5

If the minor road approach grade is an upgrade that exceeds 3 percent, add 0.2 seconds for each percent grade for left turns.

For multilane streets and highways without medians wide enough to store the design vehicle with a clearance of 3 feet on both ends of the vehicle, 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 vehicle. The median width should be included in the width of additional lanes. This is done by converting the median width to an equivalent number of 12 foot lanes.

For multilane streets and highways with medians wide enough to store the design vehicle with a clearance of 3 feet on both ends of the vehicle a two-step maneuver may be assumed. Use case B2 for crossing to the median.

3.3.9.2.4.4 Case B2 - Right Turns from the Minor Road and Case B3 - Crossing Maneuver from the Minor Road

Design time gap values for a stopped vehicle on a minor road to turn right onto or cross a two lane highway are as follows:

Design Vehicle	Time Gap (t_g) in Seconds
Passenger Car	6.5
Single Unit Truck	8.5
Combination Truck	10.5

If the approach grade is an upgrade that exceeds 3 percent, add 0.1 seconds for each percent grade.

For crossing streets and highways with more than 2 lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane to be crossed. Medians not wide enough to store the design vehicle with a clearance of 3 feet on both ends of the vehicle should be included in the width of additional lanes. This is done by converting the median width to an equivalent number of 12 foot lanes.

For crossing divided streets and highways with medians wide enough to store the design vehicle with a clearance of 3 feet on both ends of the vehicle, a two-step maneuver may be assumed. Only the number of lanes to be crossed in each step are considered.

3.3.9.2.4.5 Intersections with Traffic Signal Control (AASHTO Case D)

At signalized intersections, the first vehicle stopped on one approach should be visible to the driver of the first vehicle stopped on each of the other approaches. Left turning vehicles should have sufficient sight distance to select gaps in oncoming traffic and complete left turns. Apart from these sight conditions, no other sight triangles are needed for signalized intersections.

However, if the traffic signal is to be placed on two-way flashing operation in off peak or nighttime conditions, then the appropriate departure sight triangles for Cases B1, B2, or B3, both to the left and to the right, should be provided. In addition, if right turns on red are to be permitted, then the appropriate departure sight triangle to the left for Case B2 should be provided to accommodate right turns.

3.3.9.2.4.6 Intersections with All-Way Stop Control (AASHTO Case E)

At intersections with all-way stop control, the first stopped vehicle 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 control.

3.3.9.2.4.7 Left Turns from the Major Road (AASHTO Case F)

All locations along a major road from which vehicles are permitted to turn left across opposing traffic shall have sufficient sight distance to accommodate the left turn maneuver. In this case, the ISD is measured from the stopped position of the left turning vehicle (see **Figure 3 – 22 Sight Distance for Vehicle Turning Left from Major Road**).

Design time gap values for left turns from the major road are as follows:

Design Vehicle	Time Gap (t_g) in Seconds
Passenger Car	5.5
Single Unit Truck	6.5
Combination Truck	7.5

For left turning vehicles that cross more than one opposing lane, add 0.5 seconds for passenger cars and 0.7 seconds for trucks for each additional lane to be crossed.

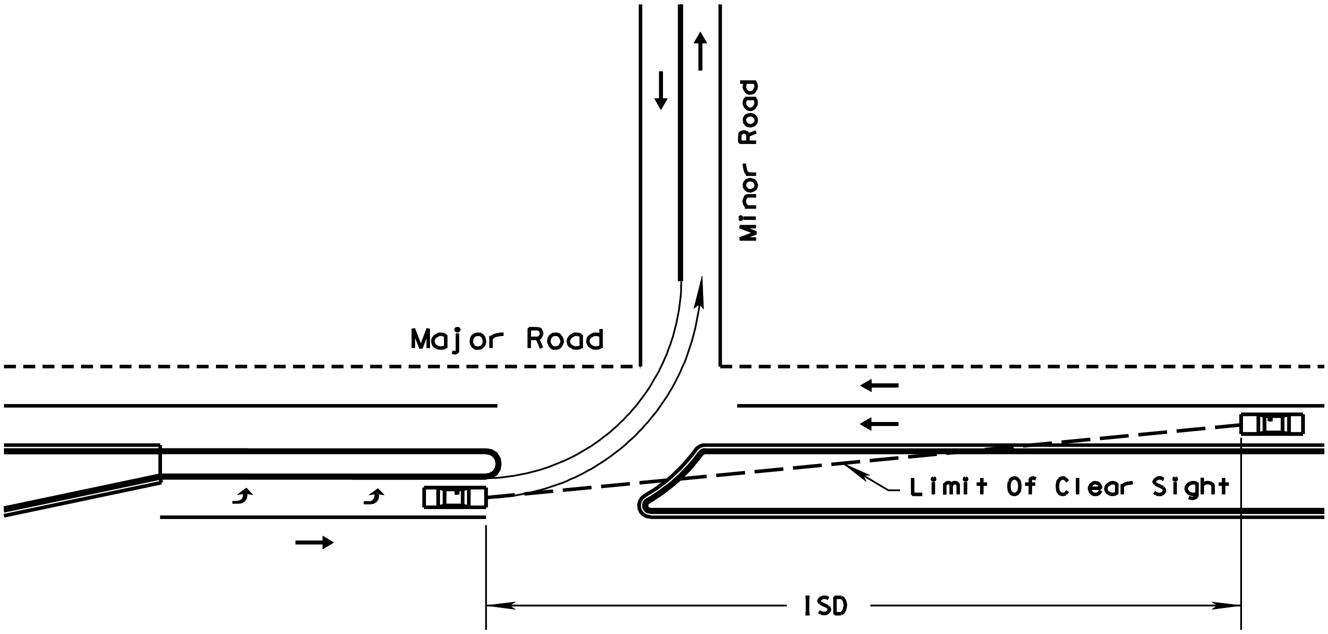
3.3.9.2.4.8 Case G - Roundabouts

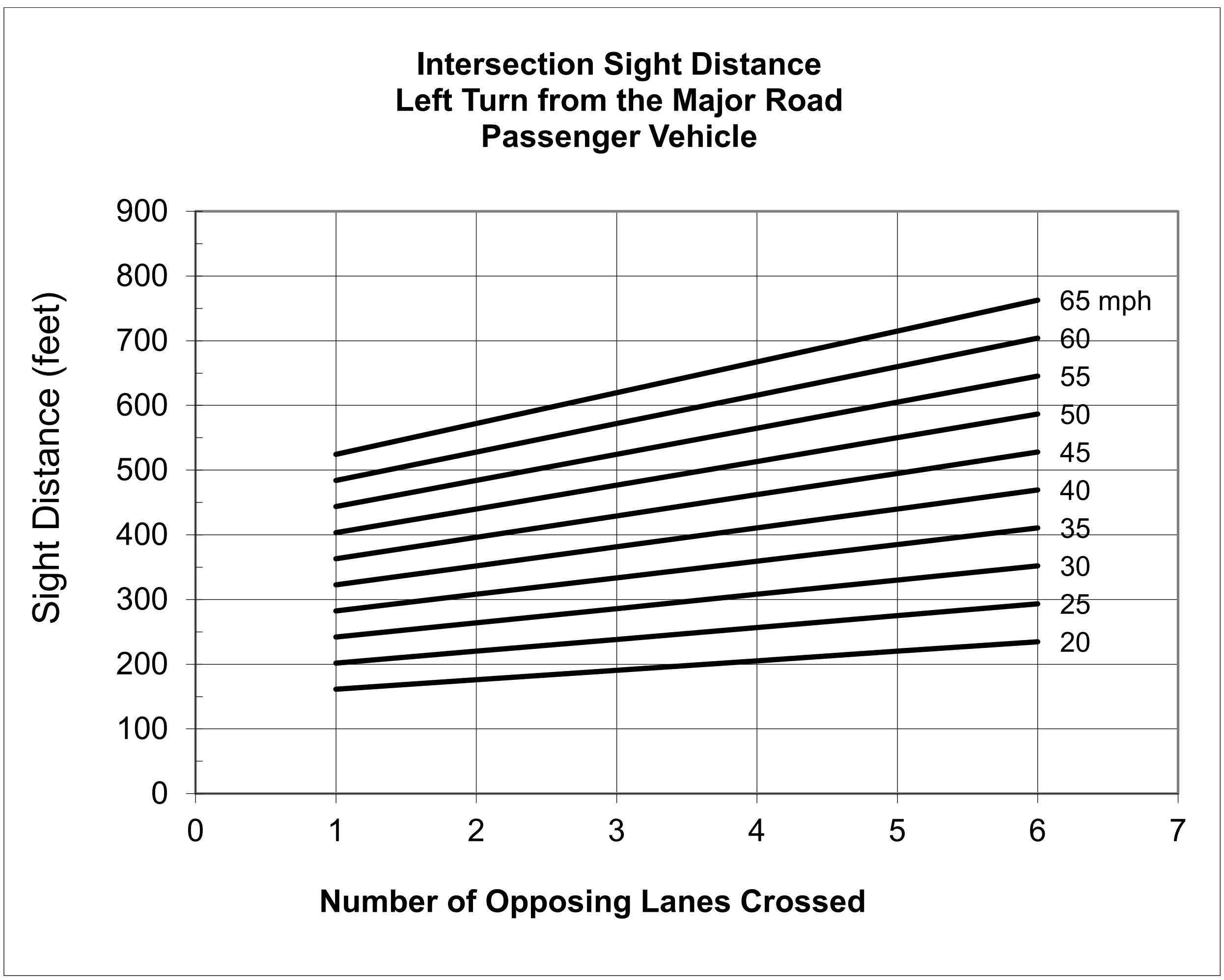
Similar in application to other intersection types, a roundabout needs intersection sight distance so that drivers on a yield-controlled approach to the roundabout can decide when to proceed into the roundabout. Drivers entering a roundabout need to see and react to the presence of potentially conflicting vehicles. Specifically, drivers entering a roundabout need to see potentially conflicting vehicles along the circulatory roadway and vehicles entering the roundabout from the immediate upstream entry. ***NCHRP Report 1043: Guide for Roundabouts***, presents a procedure for determining sight distances for use in the design of roundabouts. The report indicates that, based on international experience, it is advantageous to provide only the minimum sight distance needed at a roundabout. Additional intersection sight distance could result in higher vehicle speeds that may increase conflicts between motor vehicles, bicyclists, and pedestrians. Landscaping within the central island can be effective in restricting sight distance to the minimum needed while creating a “terminal vista” on the approach to improve visibility of the central island.

~~3.3.9.2.4.8~~3.3.9.2.4.9 Intersection Sight Distance References

The ***FDOT Design Manual, Chapter 212 Intersections***, provides ISD values for several basic intersection configurations based on Cases B1, B2, B3, and D, and may be used when applicable. For additional guidance on Intersection Sight Distance, see the ***~~AASHTO Green Book (2011)~~******2018 AASHTO Green Book***.

Figure 3-22 Sight Distance for Vehicle Turning Left from Major Road





3.3.9.3 Auxiliary Lanes

Auxiliary lanes are desirable for the safe execution of speed change maneuvers (acceleration and deceleration) and for the storage and protection of turning vehicles. Auxiliary lanes for exit or entrance turning maneuvers shall be provided in accordance with the requirements set forth in **3.3.8 Access Control**, this chapter. The pavement width and cross slopes of auxiliary lanes should meet the minimum requirements shown in **Table 3 – 21 Minimum Lane Widths**.

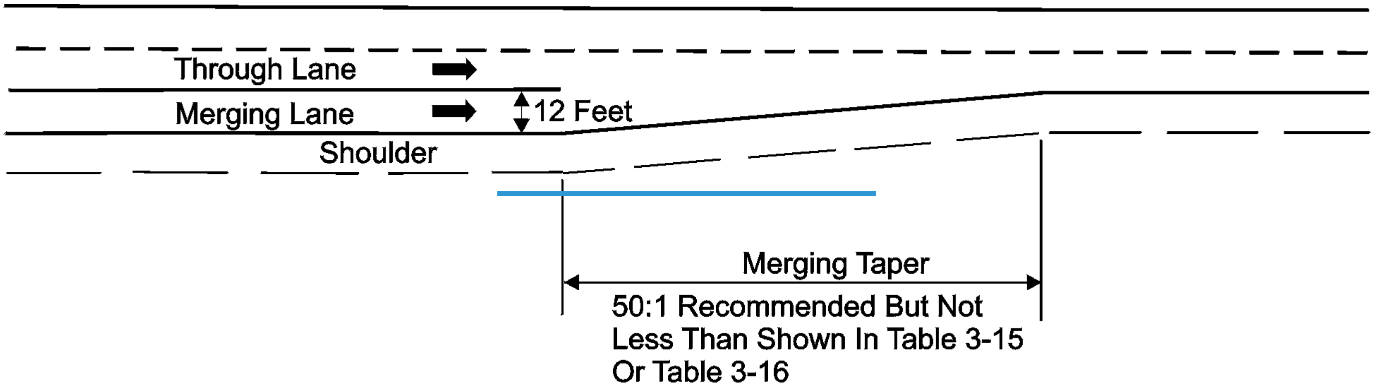
3.3.9.3.1 Merging Maneuvers

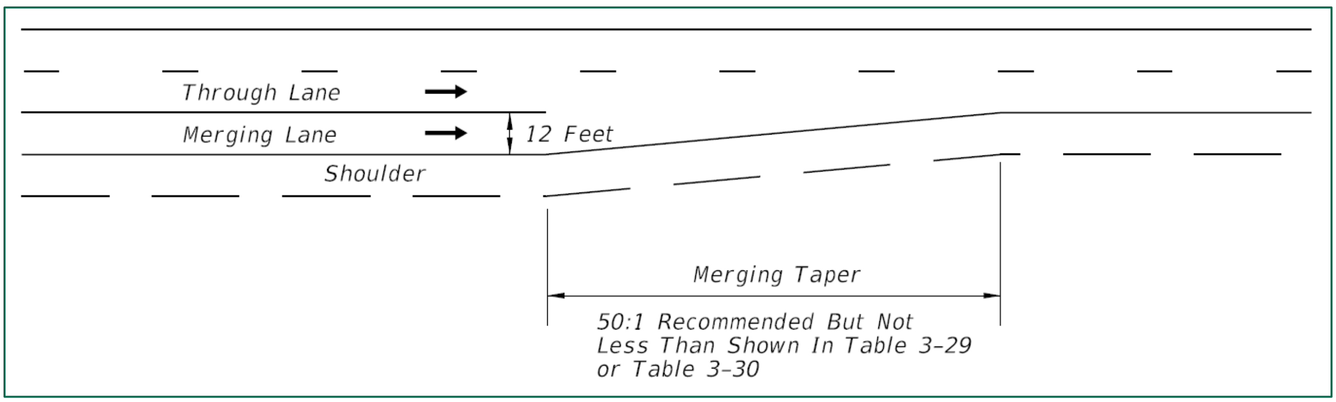
Merging maneuvers occur at the termination of climbing lanes, lane drops, entrance acceleration, and turning lanes. The location provided for this merging maneuver should, where possible, be on a tangent section of the roadway and should be of sufficient length to allow for a smooth, safe transition. The provision of ample distance for merging is essential to allow the driver time to find an acceptable gap in the through traffic and then execute a safe merging maneuver. It is recommended that a merging taper be on a 1:50 transition, but in no case, shall the length be less than set forth in **Table 3 – 29 Length of Taper for Use in Conditions with Full Width Speed Change Lanes**. The termination of this lane should be clearly visible from both the merging and through lane and should correspond to the general configuration shown in **Figure 3 – 23 Termination of Merging Lanes**. Advance warning of the merging lane termination should be provided. Lane drops shall be marked in accordance with ***Section 14-15.010, F.A.C., Manual on Uniform Traffic Control Devices (MUTCD)***.

Table 3-29 Length of Taper for Use in Conditions with Full Width Speed Change Lanes

Design Speed (mph)	20	25	30	35	40	45	50	55	60	65	70
Length of Deceleration Taper (feet)	110	130	150	170	190	210	230	250	270	290	300
Length of Acceleration Taper (feet)	80	100	120	140	160	180	210	230	250	260	280

Figure 3-23 Termination of Merging Lanes





3.3.9.3.2 Acceleration lanes

Acceleration lanes are required for all entrances to expressway and freeway ramps. Acceleration lanes may be desirable at access points to any street or highway with a large percentage of entering truck traffic.

The distance required for an acceleration maneuver is dependent on the vehicle acceleration capabilities, the grade, the initial entrance speed, and the final speed at the termination of the maneuver. The distances required for acceleration on level roadways for passenger cars are given in **Table 3 – 30 Design Lengths of Speed Change Lanes Flat Grades**. Where acceleration occurs on a grade, the required distance is obtained by using **Table 3 – 31 Ratio of Length of Speed Change Lane on Grade to Length on Level** and **Table 3 – 32 Minimum Acceleration Lengths for Entrance Terminals**.

The final speed at the end of the acceleration lane, should, desirably, be assumed as the design speed of the through roadway. The length of acceleration lane provided should be at least as long as the distance required for acceleration between the initial and final speeds. Due to the uncertainties regarding vehicle capabilities and driver behavior, additional length is desirable. The acceleration lane should be followed by a merging taper (similar to **Figure 3 – 23 Termination of Merging Lanes**), not less than that length set forth in **Table 3 – 29 Length**

of Taper for Use in Conditions with Full Width Speed Change Lanes. The termination of acceleration lanes should conform to the general configuration shown for merging lanes in **Figure 3 – 23 Termination of Merging Lanes**. Recommended acceleration lanes for freeway entrance terminals are given in **Table 3 – 32 Minimum Acceleration Lengths for Entrance Terminals**.

Table 3-30 Design Lengths of Speed Change Lanes Flat Grades - ~~2~~ 3 Percent or Less

Design Speed of turning roadway curve (mph)		Stop Condition	15	20	25	30	35	40	45	50
Minimum curve radius (feet)		---	55	100	160	230	320	430	555	695
Design Speed of Highway (mph)	Length of Taper (feet)*	Total length of DECELERATION LANE, including taper (feet)								
30	150	385	350	320	290	---	---	---	---	---
35	170	450	420	380	355	320	---	---	---	---
40	190	510	485	455	425	375	345	---	---	---
45	210	595	560	535	505	460	430	---	---	---
50	230	665	635	615	585	545	515	455	405	---
55	250	730	705	690	660	630	600	535	485	---
60	270	800	770	750	730	700	675	620	570	510
65	290	860	830	810	790	760	730	680	630	570
70	300	915	890	870	850	820	790	740	690	640
Design Speed of Highway (mph)	Length of Taper (feet)*	Total length of ACCELERATION LANE, including taper, (feet)								
30	120	300	260	---	---	---	---	---	---	---
35	140	420	360	300	---	---	---	---	---	---
40	160	520	460	430	370	280	---	---	---	---
45	180	740	670	620	560	460	340	---	---	---
50	210	930	870	820	760	660	560	340	---	---
55	230	1190	1130	1040	1010	900	780	550	380	---
60	250	1450	1390	1350	1270	1160	1050	800	670	430
65	260	1670	1610	1570	1480	1380	1260	1030	860	630
70	280	1900	1840	1800	1700	1630	1510	1280	1100	860

Note:
For urban street auxiliary lanes, shorter tapers may be used due to lower operating speeds. Refer to **Figure 3-25** for allowable taper rates.

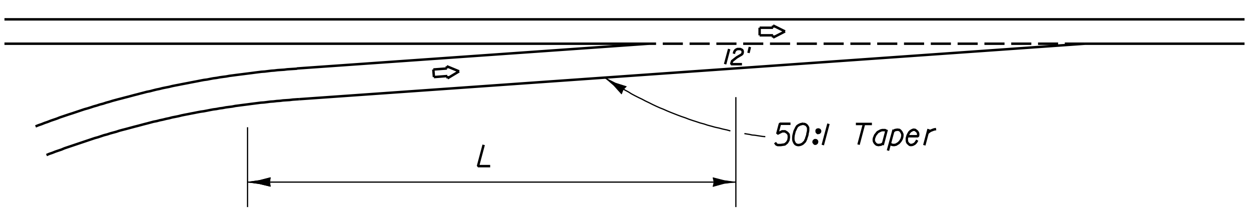
Table 3-31 Ratio of Length of Speed Change Lane on Grade to Length on Level

<table>
<tr><th colspan="3">Deceleration Lane</th><th colspan="6">Acceleration Lane</th></tr>
<tr><td></td><td colspan="2">Design Speed of Turning Roadway (mph)</td><td></td><td colspan="5">Design Speed of Turning Roadway (mph)</td></tr>
<tr><td rowspan="2">Design Speed of Highway (mph)</td><td>All Speeds</td><td>All Speeds</td><td rowspan="2">Design Speed of Highway (mph)</td><td>20</td><td>30</td><td>40</td><td>50</td><td>All Speeds</td></tr>
<tr><td>3% - 4% Upgrade</td><td>3% - 4% Downgrade</td><td colspan="4">3% -4% Upgrade</td><td>3% - 4% Upgrade</td></tr>
<tr><td rowspan="7">All Speeds</td><td rowspan="7">0.9</td><td rowspan="7">1.2</td><td>40</td><td>1.3</td><td>1.3</td><td>---</td><td>---</td><td>0.7</td></tr>
<tr><td>45</td><td>1.3</td><td>1.35</td><td>---</td><td>---</td><td>0.675</td></tr>
<tr><td>50</td><td>1.3</td><td>1.4</td><td>1.4</td><td>---</td><td>0.65</td></tr>
<tr><td>55</td><td>1.35</td><td>1.45</td><td>1.45</td><td>---</td><td>0.625</td></tr>
<tr><td>60</td><td>1.4</td><td>1.5</td><td>1.5</td><td>1.6</td><td>0.6</td></tr>
<tr><td>65</td><td>1.45</td><td>1.55</td><td>1.6</td><td>1.7</td><td>0.6</td></tr>
<tr><td>70</td><td>1.5</td><td>1.6</td><td>1.7</td><td>1.8</td><td>0.6</td></tr>
<tr><td></td><td>5% - 6% Upgrade</td><td>5% - 6% Downgrade</td><td></td><td colspan="4">5% - 6% Upgrade</td><td>5% - 6% Downgrade</td></tr>
<tr><td rowspan="7">All Speeds</td><td rowspan="7">0.8</td><td rowspan="7">1.35</td><td>40</td><td>1.5</td><td>1.5</td><td>---</td><td>---</td><td>0.6</td></tr>
<tr><td>45</td><td>1.5</td><td>1.6</td><td>---</td><td>---</td><td>0.575</td></tr>
<tr><td>50</td><td>1.5</td><td>1.7</td><td>1.9</td><td>---</td><td>0.55</td></tr>
<tr><td>55</td><td>1.6</td><td>1.8</td><td>2.05</td><td>---</td><td>0.525</td></tr>
<tr><td>60</td><td>1.7</td><td>1.9</td><td>2.2</td><td>2.5</td><td>0.5</td></tr>
<tr><td>65</td><td>1.85</td><td>2.05</td><td>2.4</td><td>2.75</td><td>0.5</td></tr>
<tr><td>70</td><td>2.0</td><td>2.2</td><td>2.6</td><td>3.0</td><td>0.5</td></tr>
<tr><td colspan="9">Note:
Ratios in this table multiplied by the values in Table 3 – 27 give the length of speed change lane for the respective grade.</td></tr>
</table>

Table 3-32 Minimum Acceleration Lengths for Entrance Terminals

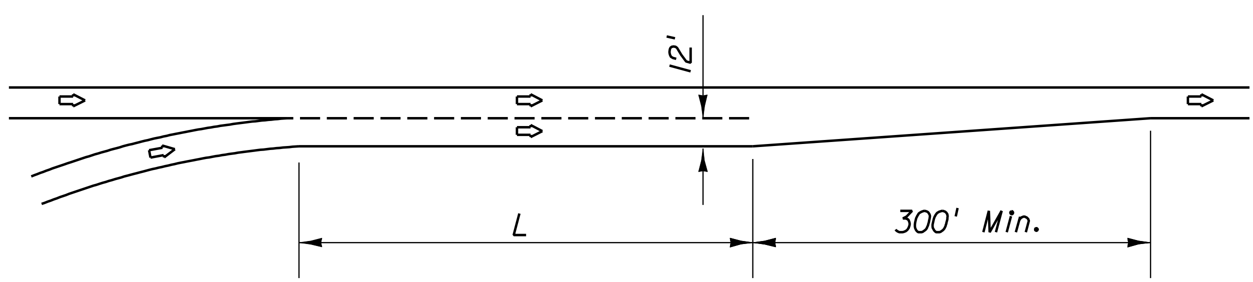
Highway Design Speed (mph)	L = Acceleration Length (feet)								
	For Entrance Curve Design Speed (mph)								
	Stop Condition	15	20	25	30	35	40	45	50
30	180	140	---	---	---	---	---	---	---
35	280	220	160	---	---	---	---	---	---
40	360	300	270	210	120	---	---	---	---
45	560	490	440	380	280	160	---	---	---
50	720	660	610	550	450	350	130	---	---
55	960	900	810	780	670	550	320	150	---
60	1200	1140	1100	1020	910	800	550	420	180
65	1410	1350	1310	1220	1120	1000	770	600	370
70	1620	1560	1520	1420	1350	1,230	1000	820	580

Expressway and Freeway Entrance Terminals



TAPER TYPE

Recommended when design speed at entrance curve is 50 mph or greater.



PARALLEL TYPE

Recommended when design speed at entrance curve is less than 50 mph.

3.3.9.3.3 Exit Lanes

Auxiliary lanes for exiting maneuvers provide space outside the through lanes for protection and storage of decelerating vehicles exiting the facility.

- Deceleration Lanes - The primary function of deceleration lanes is to provide a safe travel path for vehicles decelerating from the operating speed on the through lanes. Deceleration lanes are required for all freeway exits and are desirable on high-speed (design speed greater than 50 mph) streets and highways.

The distance required for deceleration of passenger cars is given in **Table 3 – 33 Minimum Deceleration Lengths for Exit Terminals**.

The required distance for deceleration on grades is given in **Tables 3 – 30 Design Lengths of Speed Change Lanes Flat Grades - 2 Percent or Less** and **3 – 31 Ratio of Length of Speed Change Lane on Grade to Length on Level**.

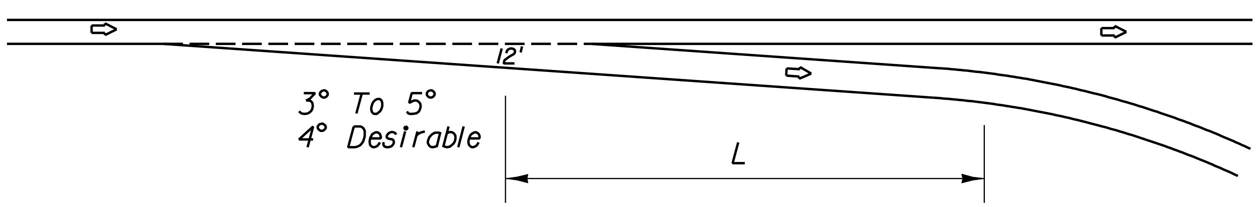
The length of deceleration lanes shall be no less than the values obtained from **Tables 3 – 30 and 3 – 31** and should be increased wherever feasible. The initial speed should, desirably, be taken as the design speed of the highway. The final speed should be the design speed at the exit (e.g., a turning roadway) or zero, if the deceleration lane terminates at a stop or traffic signal. A reduction in the final speed to be used is particularly important if the exit traffic volume is high since the speed of these vehicles may be significantly reduced.

The entrance to deceleration (and climbing) lanes should conform to the general configuration shown in **Figure 3 – 24 Entrance for Deceleration Lane**. The initial length of straight taper, shown in **Table 3 – 32 Minimum Acceleration Lengths for Entrance Terminals**, may be utilized as a portion of the total required deceleration distance. The pavement surface of the deceleration lane should be clearly visible to approaching traffic, so drivers are aware of the maneuvers required. Recommended deceleration lanes for exit terminals are given in **Table 3 – 33 Minimum Deceleration Lengths for Exit Terminals**.

Table 3-33 Minimum Deceleration Lengths for Exit Terminals

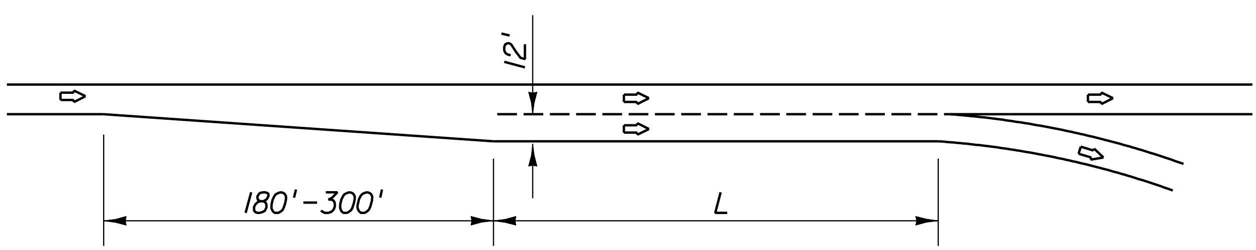
Highway Design Speed (mph)	L = Deceleration Length (feet)								
	For Design Speed of Exit Curve (mph)								
	Stop Condition	15	20	25	30	35	40	45	50
30	235	200	170	140	---	---	---	---	---
35	280	250	210	185	150	---	---	---	---
40	320	295	265	235	185	155	---	---	---
45	385	350	325	295	250	220	---	---	---
50	435	405	385	355	315	285	225	175	---
55	480	455	440	410	380	350	285	235	---
60	530	500	480	460	430	405	350	300	240
65	570	540	520	500	470	440	390	340	280
70	615	590	570	550	520	490	440	390	340

Expressway and Freeway Exit Terminals



TAPER TYPE

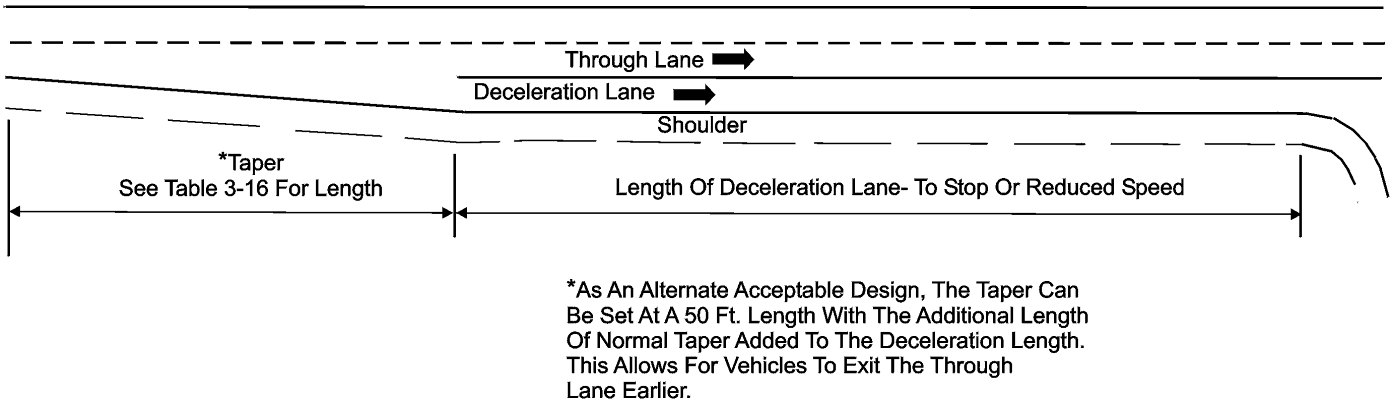
Recommended when design speed at exit curve is 50 mph or greater and when approach visibility is good.



PARALLEL TYPE

Recommended when design speed at exit curve is less than 50 mph or when approach visibility is not good.

Figure 3-24 Entrance for Deceleration Lane



3.3.9.3.4 Auxiliary Lanes at Intersections

The primary function of auxiliary lanes at intersections is to accommodate speed changes and maneuvering of turning traffic. They are typically added to increase capacity and/or reduce crashes at an intersection. Auxiliary lanes for deceleration and storage of queuing vehicles are used preceding intersections and median openings for left-turning and right-turning movements. In some cases, auxiliary lanes for acceleration are used following right-turning movements.

3.3.9.3.4.1 Widths of Auxiliary Lanes

The minimum widths for auxiliary lanes are given in **Table 3 – 21 Minimum Lane Widths**.

3.3.9.3.4.2 Lengths of Auxiliary Lanes for Deceleration

Recommended lengths for auxiliary lanes for deceleration (turn lanes) at intersections are provided in **Figure 3 – 25 Auxiliary Lanes for Deceleration at Intersections (Turn Lanes)** and **Table 3 – 34 Turn Lanes – Curbed and Uncurbed Medians**. These lengths are based on FDOT criteria. As shown in **Figure 3 – 25** the total length of turn lanes consists of three components, (1) Deceleration Length, (2) Storage or Queue Length and (3) Entering Taper. It is common practice to accept a moderate amount of deceleration within the through lanes and to consider the taper as part of the deceleration length. The length criteria for each of the auxiliary lane components are explained as follows:

- **Deceleration Length**
    - The required total deceleration length is that needed for a safe and comfortable stop from the design speed of the highway. Minimum deceleration lengths (including taper) for auxiliary lanes are provided in **Figure 3 – 25** and are based on minimum stopping sight distance.

- **Storage (Queue) Length**
    - The auxiliary lane should be sufficiently long to store the number of vehicles likely to accumulate during a critical period. The storage length should be sufficient to avoid the possibilities of turning vehicles stopping in the through lanes or the entrance to the auxiliary lane being blocked by vehicles queuing in the through lanes.
    - At unsignalized intersections the storage length, exclusive of taper, may be based on the number of turning vehicles likely to arrive in an average two-minute period within the peak hour. For low volume intersections where a traffic study is not justified, a minimum 50-foot queue length (2 vehicles) should be provided on rural highways. A minimum 100-foot queue length (4 vehicles) should be provided in urban areas. Locations with over 10% truck traffic should accommodate at least one car and one truck.
    - At signalized intersections, the required storage length is determined by traffic study and depends on the signal cycle length, the signal phasing arrangement and the rate of arrivals and departures of turning vehicles. The storage length is a function of the probability of occurrence of events and should be based on 1.5 to 2 times the average number of vehicles that would store per cycle that is predicted in the design volume.
    - Where dual turning lanes are used, the required storage length is reduced to approximately one-half of that required for single-lane operation.
- **Approach End Taper**
    - The FDOT's criteria for approach end taper lengths for turn lanes are 50 feet for a single turn lane and 100 feet for a double turn lane, as shown in **Figure 3 - 25 Auxiliary Lanes for Deceleration at Intersections (Turn Lanes)** and **Table 3 – 34 Turn Lanes – Curbed and Uncurbed Medians**. These taper lengths apply to all design speeds and are recommended for use on turn lanes on all roads. Short taper lengths are intended to provide approaching road users with positive identification of an added auxiliary lane and results in a longer full width auxiliary lane than use of longer taper lengths based on the path that road users actually follow. The clearance distances L_1 and L_3 account for the full transition lengths a road user will use to enter the auxiliary lane for various speed conditions assumed for design.
    - It is acceptable to lengthen the taper up to L_1 for single left turns and L_3 for double left turns where traffic study can establish that left turn queue vehicles are adequately provided for within the design queue length and through vehicle queues will not block access to the left turn lane(s).

This is a working document that has not been adopted.

Figure 3-25 Auxiliary Lanes for Deceleration at Intersections (Turn Lanes)

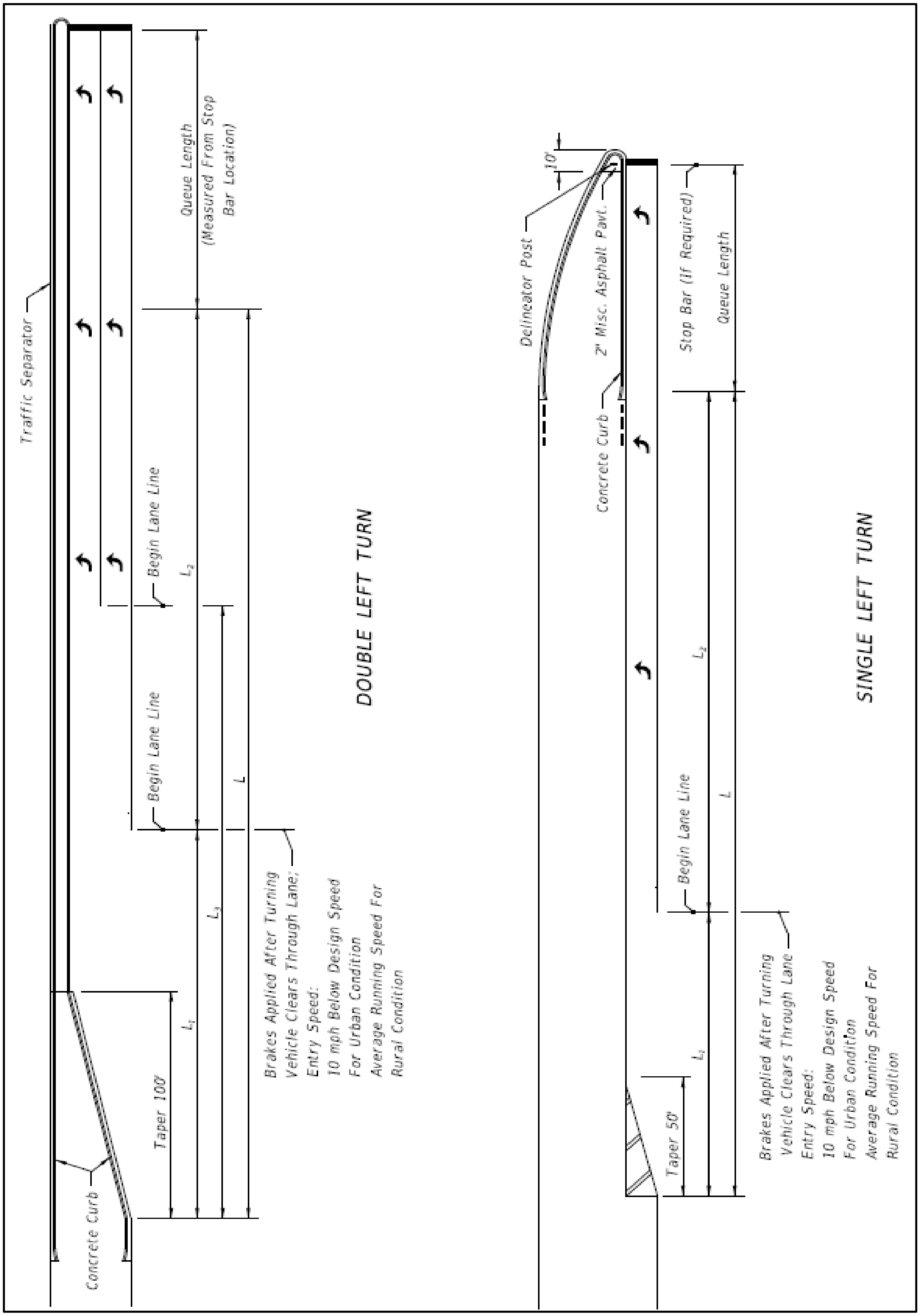


Table 3-34 Turn Lanes – Curbed and Uncurbed Medians

Design Speed (mph)	Entry Speed (mph)	Clearance Distance L_1 (feet)	Urban Conditions			Rural Conditions		
			Brake to Stop Distance L_2 (feet)	Total Decel. Distance L (feet)	Clearance Distance L_3 (feet)	Brake to Stop Distance L_2 (feet)	Total Decel. Distance L (feet)	Clearance Distance L_3 (feet)
≤ 30	≤ ~~25~~20	~~60~~70	~~75~~50	~~135~~120	100			
35	25	70	75	145	110	----	----	----
40	30	80	75	155	120	----	----	----
45	35	85	100	185	135	----	----	----
50	~~40/~~44	105	~~135~~----	~~240~~----	~~160~~----	185	290	160
55	48	125	----	----	----	225	350	195
60	52	145	----	----	----	260	405	230
65	55	170	----	----	----	290	460	270

Note:
Right turn lane tapers and distances are identical to left turn lanes under stop control conditions. For free flow or yield control conditions, taper lengths and distances are site specific.

3.3.9.3.4.3 Lengths of Auxiliary Lanes for Acceleration

Acceleration lanes similar to those used for freeways and expressways are sometimes used at intersections. They are not always desirable at stop-controlled intersections where entering drivers can wait for an opportunity to merge without disrupting through traffic. Acceleration lanes are advantageous on roads without stop control and on all high-volume roads even with stop control where openings between vehicles in the peak-hour traffic streams are infrequent and short. When used, acceleration lanes at intersections should be designed using the criteria provided in **Section 3.3.9.3.2 Acceleration Lanes**.

3.3.9.4 Turning Roadways at Intersections

The design and construction of turning roadways shall meet the same general requirements for through roadways, except for the specific requirements given in the subsequent sections.

3.3.9.4.1 Design Speed

Lanes for turning movements at grade intersections may, where justified, be based on a design speed as low as 10 mph. Turning roadways with design speeds in excess of 40 mph shall be designed in accordance with the requirements for through roadways.

A variable design speed may be used to establish cross section and alignment criteria for turning roadways that will experience acceleration and deceleration maneuvers.

3.3.9.4.2 Horizontal Alignment

- Curvature - The minimum permitted radii (maximum degree) of curvature for various values of superelevation are given in **Table 3 – 35 Superelevation Rates for Curves at Intersections**. These should be considered as minimum values only and the radius of curvature should be increased wherever feasible. Further information contained in AASHTO – "**A Policy on Geometric Design of Highways and Streets" - ~~2011~~ 2018**, should also be considered.

Table 3-35 Superelevation Rates for Curves at Intersections

	Design Speed (mph)					
	20	**25**	**30**	**35**	**40**	**45**
Minimum Superelevation Rate	0.02	0.04	0.06	0.08	0.09	0.10
Minimum Radius (feet)	90	150	230	310	430	540

The rate of 0.02 is considered the practical minimum for effective drainage across the surface.

Notes:
Preferably use superelevation rates greater than these minimum values.

- Superelevation Transition - Minimum superelevation transition (runoff) rates (maximum relative gradients) are given in **Tables 3 – ~~35~~ 36 Maximum Rate of Change in Pavement Edge Elevation for Curves at Intersections** and **3 – 36 Maximum Algebraic Difference in Pavement Cross Slope at Turning Roadway Terminals**. Other information given in AASHTO – "**A Policy on Geometric Design of Highways and Streets" - ~~2011~~2018**, should also be considered.

Table 3-36 Maximum Rate of Change in Pavement Edge Elevation for Curves at Intersections

Design Speed (mph)	20	25	30	35	40	45	50	55	60	65	70
Maximum relative gradients for profiles between the edge of two lane pavement and the centerline (percent)	0.74	0.70	0.66	0.62	0.58	0.54	0.50	0.47	0.45	0.43	0.40

Table 3-37 Maximum Algebraic Difference in Pavement Cross Slope at Turning Roadway Terminals

Design Speed of Exit or Entrance Curve (mph)	Maximum Algebraic Difference in Cross Slope at Crossover Line (percent)
20 and under	5.0 to 8.0
25 and 30	5.0 to 6.0
35 and over	4.0 to 5.0

3.3.9.4.3 Vertical Alignment

Grades on turning roadways should be as flat as practical and long vertical curves should be used wherever feasible. The length of vertical curves shall be no less than necessary to provide minimum stopping sight distance. Minimum stopping sight distance values are given in **Table 3 – 4 Minimum Stopping Sight Distances**. For additional guidance on vertical alignment for turning roadways, see AASHTO – **"A Policy on Geometric Design of Highways and Streets" - ~~2011~~ 2018**.

3.3.9.4.4 Cross Section Elements

- Number of Lanes - One-way turning roadways are often limited to a single traffic lane. In this case, the total width of the roadway shall be sufficient to allow traffic to pass a disabled vehicle. Two-way, undivided turning roadways should be avoided. Medians or barriers should be utilized to separate opposing traffic on turning roadways.
- Lane Width - The width of all traffic lanes should be sufficient to accommodate (with adequate clearances) the turning movements of the expected types of vehicles. The minimum required lane widths for turning roadways are given in **Table 3 - 38 Derived Pavement Widths for Turning Roadways for Different Design Vehicles**. Changes in lane widths should be gradual and should be accomplished in coordination with adequate transitions in horizontal curvature.
- Shoulders - On one-lane turning roadways, serving expressways and other arterials (e.g., loops, ramps), the right hand shoulder should be at least 6 feet wide. The left hand shoulder should be at least 6 feet wide in all cases. On two-lane, one-way roadways, both shoulders should be at least 6 feet wide. Where guardrails or other barriers are used, they should be placed at least 8 feet from edge of travel lane. Guardrails should be placed 2 feet outside the normal shoulder width.
- Clear Zones - Turning roadways should, as a minimum, meet all open highway criteria for clear zones on both sides of the roadway. The areas on the outside of curves should be

wider and more gently sloped than the minimum values for open highways. Guardrails or similar barriers shall be used if the minimum width and slope requirements cannot be obtained.

Further criteria and requirements for roadway design are given in **Chapter 4 - Roadside Design**.

Table 3-38 Derived Pavement Widths for Turning Roadways for Different Design Vehicles

Radius on Inner Edge of Pavement, R (feet)	Case 1, One-Lane Operation, No Provision for Passing a Stalled Vehicle												
	P	SU-30	Su-40	City Bus	S-Bus-36	A-Bus	WB-40	WB-62	WB-67	WB-67D	MH	P/T	P/B
50	13	18	21	21	~~18~~19	22	23	44	57	29	18	19	18
75	13	17	18	19	17	19	20	30	33	23	17	17	17
100	13	16	17	18	16	18	18	25	28	21	16	16	16
150	12	15	16	17	16	17	17	22	23	19	15	16	15
200	12	15	16	16	15	16	16	20	21	18	15	15	15
300	12	15	15	16	15	16	15	18	19	17	15	15	15
400	12	15	15	15	15	15	15	17	18	16	15	15	14
500	12	14	15	15	14	15	15	17	17	16	14	14	14
Tangent	12	14	14	15	14	15	14	15	15	15	14	14	14

This is a working document that has not been adopted.

Table 3-38 Derived Pavement Widths for Turning Roadways for Different Design Vehicles (Continued)

Radius on Inner Edge of Pavement, R (feet)	Case II, One-Lane, One-Way Operation, with Provision for Passing a Stalled Vehicle by Another of the Same Type												
	P	SU-30	Su-40	City Bus	S-Bus-36	A-Bus	WB-40	WB-62	WB-67	WB-67D	MH	P/T	P/B
50	20	30	36	38	31	40	39	81	109	50	30	30	28
75	19	27	30	32	27	34	32	53	59	39	27	27	26
100	18	25	27	~~30~~29	25	30	29	44	48	34	25	25	24
150	18	23	25	27	23	27	26	36	38	29	23	23	23
200	17	22	24	25	23	26	24	32	34	27	22	22	22
300	17	22	22	24	22	24	23	28	30	25	22	22	21
400	17	21	22	23	21	23	22	26	27	24	21	21	21
500	17	21	21	23	21	23	22	25	26	23	21	21	21
Tangent	~~12~~17	~~14~~20	~~14~~20	~~15~~21	~~14~~20	~~15~~21	~~14~~20	~~15~~21	~~15~~21	~~15~~21	~~14~~20	~~14~~20	~~14~~20

Radius on Inner Edge of Pavement, R (feet)	Case III, Two-Lane Operation, Either One- or Two-Way (Same Type Vehicle in Both Lanes)												
	P	SU-30	Su-40	City Bus	S-Bus-36	A-Bus	WB-40	WB-62	WB-67	WB-67D	MH	P/T	P/B
50	26	36	42	44	37	46	45	87	115	56	36	36	34
75	25	33	36	38	33	40	38	59	65	45	33	33	32
100	24	31	33	35	31	36	35	50	54	40	31	31	30
150	24	29	31	33	29	33	32	42	44	35	29	29	29
200	23	28	30	31	29	32	30	38	40	33	28	28	28
300	23	28	28	30	28	30	29	34	36	31	28	28	27
400	23	27	28	29	27	29	28	32	33	30	27	27	27
500	23	27	27	29	27	29	28	31	32	29	27	27	27
Tangent	23	26	26	27	26	27	28	27	27	27	26	26	26

Source – ~~2011~~ 2018 AASHTO Greenbook, Table 3-~~28b~~ 26a Derived Pavement Widths for Turning Roadways for Different Design Vehicle

3.3.9.5 At Grade Intersections

3.3.9.5.1 Turning Radii

Where right turns from through or turn lanes will be negotiated at low speeds (less than 10 mph), the minimum turning capabilities of the vehicle may govern the design. It is desirable that the turning radius and the required lane width be provided in accordance with the criteria for turning roadways. The radius of the inside edge of traveled way should be sufficient to allow the expected vehicles to negotiate the turn without encroaching the shoulder or adjacent traffic lanes.

Where turning roadway criteria are not used, the radius of the inside edge of traveled way should be no less than 25 feet. The use of three-centered compound curves is also a reasonable practice to allow for transition into and out of the curve. The recommended radii and arrangement of compound curves instead of a single simple curve is given in AASHTO – "**A Policy on Geometric Design of Highways and Streets" - ~~2011~~ 2018**.

3.3.9.5.2 Cross Section Correlation

The correlation of the cross section of two intersecting roadways is frequently difficult. A careful analysis should be conducted to ensure changes in slope are not excessive and adequate drainage is provided. At stop-controlled intersections, the through roadway cross section should be carried through the intersection without interruption. Minor roadways should approach the intersection at a slightly reduced elevation so the through roadway cross section is not disturbed. At signalized intersections, it is sometimes necessary to remove part of the crown in order to avoid an undesirable hump in one roadway.

Intersections of grade or cross slope should be gently rounded to improve vehicle operation. Pavement generally should be sloped toward the intersection corners to provide superelevation for turning maneuvers and to promote proper drainage.

Where islands are used for channelization, the width of traffic lanes for turning movements shall be no less than the widths recommended by AASHTO.

3.3.9.5.3 Median Openings

Median openings should be restricted in accordance with the requirements presented in **3.3.8 Access Control**, this chapter. Where a median opening is required, the length of the opening shall be no less than 40 feet. Median curbs should be terminated gradually without the exposure of abrupt curb ends. The termination requirements are given in **Chapter 4 - Roadside Design**.

3.3.9.5.4 Channelization

Channelization of at grade intersections is the regulation or separation of conflicting movements into definite travel paths by islands, markings, or other means, to promote safe, orderly traffic flow. The major objective of channelization is to clearly define the appropriate paths of travel and thus assist in the prevention of vehicles deviating excessively or making wrong maneuvers. Channelization may be used effectively to define the proper path for exits, entrances, and intersection turning movements. The methods used for channelization should be as simple as possible and consistent in nature. The channelized intersection should appear open and natural to the approaching driver. Channelization should be informative rather than restrictive in nature.

The use of low sloping curbs and flush medians and islands can provide adequate delineation in most cases. Islands should be clearly visible and, in general, should not be smaller than 100 square feet in area. The use of small and/or numerous islands should be avoided.

Pavement markings are a useful and effective tool for providing delineation and channelization in an informative rather than restrictive fashion. The layout of all traffic control devices should be closely coordinated with the design of all channelization.

3.3.9.6 Driveways

Direct driveway access within the area of influence of the intersection should be discouraged.

Driveways from major traffic generators (greater than 400 vpd), or those with significant truck/bus traffic, should be designed as normal intersections.

3.3.9.7 Interchanges

The design of interchanges for the intersection of a freeway with a major street or highway, collector/distributor road, or other freeway is a complex problem. The location and spacing of intersections should follow the requirements presented in **3.3.8 Access Control**, this chapter. The design of interchanges shall follow the general intersection requirements for deceleration, acceleration, merging maneuvers, turning roadways, and sight distance.

Interchanges, particularly along a given freeway, should be reasonably consistent in their design. A basic principle in the design should be to develop simple open interchanges that are easily traversed and understandable to the driver. Complex interchanges with a profusion of possible travel paths are confusing and hazardous to the motorist and are generally inefficient.

Intersections with minor streets or highways or collector/distributor roads may be accomplished by simple diamond interchanges. The intersection of exit and entrance ramps with the crossroad shall meet all intersection requirements.

The design of freeway exits should conform to the general configurations given in **Table 3 – 33 Minimum Deceleration Lengths for Exit Terminals**. Exits should be on the right and should be placed on horizontal curves. Where deceleration on an exit loop is required, the deceleration alignment should be designed so the driver receives adequate warning of the approaching increase in curvature. This is best accomplished by gradually increasing the curvature and the resulting centrifugal force. This increasing centrifugal force provides warning to the driver that he must slow down. A clear view of the exit loop should also be provided. The length of deceleration shall be no less than the values shown in **Table 3 – 30**.

Entrances to freeways should be designed in accordance with the general configurations shown below **Table 3 – 32 Minimum Acceleration Lengths for Entrance Terminals**. Special care should be taken to ensure vehicles entering from loops are not directed across through travel lanes. The entering roadway should be brought parallel (or nearly so) to the through lanes before entry is permitted. Where acceleration is required, the distances shown in **Table 3 – 32** shall, as a minimum, be provided. Exits and entrances to all high-speed facilities (design speed greater than 50 mph), should, where feasible, be designed in accordance with **Tables 3 – 32** and **3 – 33**. The lengths obtained from **Tables 3 – 32** and **3 –33** should be adjusted for grade by using the ratios in **Table 3 – 31**.

The selection of the type and exact design details of a particular interchange requires considerable study and thought. The guidelines and design details given in AASHTO **"A Policy on Geometric Design of Highways and Streets" - ~~2011~~ 2018**, should generally be considered as minimum criteria.

3.3.9.8 Clear Zones

The provisions of ample clear zone or proper redirection of energy absorbing devices is particularly important at intersections. Every effort should be made to open up the area around the intersection to provide adequate clear zone for vehicles that have left the traveled way. Drivers frequently leave the proper travel path due to unsuccessful turning maneuvers or due to the necessity for emergency avoidance maneuvers. Vehicles also leave the roadway after intersection collisions and roadside objects should be removed to reduce the probability of second impacts. The roadside areas at all intersections and interchanges should be contoured to provide shallow slopes and gentle changes in grade.

The roadside clear zone of intersecting roadways should be carried throughout intersections with no discontinuities or interruptions. Poles and support structures for lights, signs, and signals should not be placed in medians or within the roadside clear zone.

The design of guardrails or other barriers should receive particular attention at intersections. Impact attenuators should be used in all gore and other areas where structures cannot be removed.

Particular attention should be given to the protection of pedestrians in intersection areas - **Chapter 8 - Pedestrian Facilities**. Further criteria and requirements for clear zone and protection devices at intersections are given in **Chapter 4 - Roadside Design**.

3.3.10 Other Design Factors

3.3.10.1 Pedestrian Facilities

The layout and design of the street and highway network should include provisions for pedestrian traffic in urban areas. All pedestrian crossings and pathways within the road right of way should be considered and designed as in integral part of any street or urban highway.

3.3.10.1.1 Policy and Objectives - New Facilities

The planning and design of new streets and highways shall include provisions for the safe, orderly movement of pedestrian traffic.

The overall objective is to provide a safe, continuous, convenient, and comfortable trip for pedestrian traffic.

3.3.10.1.2 Accessibility Requirements

Pedestrian facilities, such as sidewalks, shared use paths, underpasses, overpasses, and transit boarding and alighting areas shall be designed to accommodate people with disabilities. In addition to the design criteria provided in this Manual, the ***United States Department of Transportation ADA Standards for Transportation Facilities (2006)*** and ***Department of Justice ADA Standards (2010)*** as required by 49 C.F.R 37.41 or 37.43; and the ***2020 Florida Building Code – Accessibility, 7th Edition*** as required by ***Rule Chapter 61G20-4.002, Florida Administrative Code*** impose additional requirements for the design and construction of pedestrian facilities. The ***Proposed Public Rights-of-Way Accessibility Guidelines (PROWAG)*** provides additional information on the design of accessible pedestrian facilities.

3.3.10.1.3 Sidewalks and Shared Use Paths

Sidewalks should provide a safe, comfortable space for pedestrians. The width of sidewalks is dependent upon the roadside environment, volume of pedestrians, and the presence of businesses, schools, parks, and other pedestrian attractors. The minimum and recommended widths for sidewalks and shared use paths is covered in **Chapter 8 – Pedestrian Facilities** and **Chapter 9 – Bicycle Facilities**. To ensure compliance with federal and state accessibility requirements for sidewalks :

- Sidewalks less than 60 inches wide must have passing spaces of at least 60 inches by 60 inches, at intervals not to exceed 200 feet.
- The minimum clear width may be reduced to 32 inches for a short distance. This distance must be less than 24 inches long, and separated by 5-foot long sections with 48 inches of clear width.
- Sidewalks not constrained within the roadway right of way with slopes greater than 1:20 are considered ramps and must be designed as such.

3.3.10.2 Bicycle Facilities

Provisions for bicycle traffic should be incorporated into the street or highway design. All new roadways and major corridor improvements, except limited access highways, should be designed and constructed under the assumption they will be used by bicyclists. Roadway conditions should be favorable for bicycling. This includes appropriate drainage grates, pavement markings, and railroad crossings, smooth pavements, and signals responsive to bicycles. In addition, facilities such as bicycle lanes, shared use paths, and paved shoulders, should be included to the fullest extent feasible. All flush shoulder arterial and collector roadway sections should be given consideration for the construction of 4-foot or 5-foot paved shoulders. In addition, all curbed arterial and collector sections should be given consideration for bicycle lanes.

For additional information on bicycle facilities design and the design of shared use paths, refer to **Chapter 9 – Bicycle Facilities**.

3.3.10.3 Bridge Design Loadings

The minimum design loading for all new and reconstructed bridges shall be in accordance with **Chapter 17 – Bridges and Other Structures**.

3.3.10.4 Dead End Streets and Cul-de-Sacs

The end of a dead-end street should permit travel return with a turnaround area, considering backing movements, which will accommodate single truck or transit vehicles without encroachment upon private property. Recommended treatment for dead end streets and cul-de-sacs is given in **Figure 5-1 Types of Cul-de-Sacs and Dead-End Streets** of AASHTO – **"A Policy on Geometric Design of Highways and Streets" - ~~2011~~ 2018**.

3.3.10.5 Bus Benches and Transit Shelters

Bus benches should be set back at least 10 feet from the travel lane in curbed sections with a design speed of 45 mph or less, and outside the clear zone in flush shoulder sections. See **Chapter 4 – Roadside Design, Table 4 – 2 Lateral Offset** for further information.

Any bus bench or transit shelter adjacent to a sidewalk within the right of way of any street or highway shall be located so as to leave at least 48 inches of clearance for pedestrians and persons in wheelchairs. An additional one foot of clearance is required when any side of the sidewalk is adjacent to a curb or barrier. Such clearance shall be measured in a direction perpendicular to the centerline of the road. A separate bench pad or sidewalk flare out that provides a 30-inch-wide by 48-inch-deep wheelchair space adjacent to the bench shall be provided. Transit shelters should be set back, rather than eliminated during roadway widening.

Additional information on the design of transit facilities is found in **Chapter 13 – Public Transit**.

3.3.10.6 Traffic Calming

Often there are community concerns with controlling travel speeds impacting the safety of a street or highway such as in areas of concentrated pedestrian activities, those with narrow right of way, areas with numerous access points, on street parking, and other similar concerns. Local authorities may elect to use traffic calming design features that could include, but not be limited to, the installation of speed humps, speed tables, chicanes, or other pavement undulations. Roundabouts are also another method of dealing with this issue at intersections. For additional details and traffic calming treatments, refer to **Chapter 15 – Traffic Calming**.

3.3.11 Reconstruction

3.3.11.1 Introduction

The reconstruction (improvement or upgrading) of existing facilities may generate equal or greater safety benefits than similar expenditures for the construction of new streets and highways. Modifications to increase capacity should be evaluated for the potential effect on the highway safety characteristics. The long-range objectives should be to bring the existing network into compliance with current standards.

3.3.11.2 Evaluation of Streets and Highways

The evaluation of the safety characteristics of streets and highways should be directed towards the identification of undesirable features on the existing system. Particular effort should be exerted to identify the location and nature of features with a high crash potential. Methods for identifying and evaluating hazards include the following:

- Identification of any geometric design feature not in compliance with minimum or desirable standards. This could be accomplished through a systematic survey and evaluation of existing facilities.
- Review of conflict points along a corridor.
- Information from maintenance or other personnel.
- Review of crash reports and traffic counts to identify locations with a large number of crashes or a high crash rate.
- Review for expected pedestrian and bicycle needs.

3.3.11.3 Priorities

A large percentage of street and highway reconstruction and improvements is directed toward increasing efficiency and capacity. The program of reconstruction should be based, to a large extent, upon priorities for the improvement of safety characteristics.

The priorities for safety improvements should be based on the objective of obtaining the maximum reduction in crash potential for a given expenditure of funds. Elimination of conditions that may result in serious or fatal crashes should receive the highest priority in the schedule for reconstruction.

Specific high priority problem areas that should be corrected by reconstruction include the following:

- Obstructions to sight distance which can be economically corrected. The removal of buildings, parked vehicles, vegetation, large poles or groups of poles that significantly reduce the field of vision, and signs to improve sight distance on curves and particularly at intersections, can be of immense benefit in reducing crashes. The purchase of required line of sight easements is often a wise expenditure of highway funds. The establishment of sight distance setback lines is encouraged.

- Roadside and median hazards which can often be removed or relocated farther from the traveled way. Where removal is not feasible, objects should be shielded by redirection or energy absorbing devices. The reduction of the roadside hazard problem generally provides a good return on the safety dollar. Details and priorities for roadside hazard reduction, which are presented in **Chapter 4 – Roadside Design**, should be incorporated into the overall priorities of the reconstruction program.
- Poor pavement surfaces which have become hazardous should be maintained or reconstructed in accordance with the design criteria set forth in **Chapter 5 – Pavement Design And Construction**, and **Chapter 10 – Maintenance And Resurfacing**.
- Specific design features which could be applied during reconstruction to enhance the operations and safety characteristics of a roadway include the following:
- Addition of lighting.
- Frontage roads may be utilized to improve the efficiency and safety of streets and highways with poor control of access.
- Widening of pavements and shoulders. This is often an economically feasible method of increasing capacity and reducing traffic hazards. Provision of median barriers (**Chapter 4 - Roadside Design**) can also produce significant safety benefits.
- The removal, streamlining, or modification of drainage structures.
- Alignment modifications are usually extensive and require extensive reconstruction of the roadway. Removal of isolated sharp curves is a reasonable and logical step in alignment modification. If major realignment is to be undertaken, every effort should be made to bring the entire facility into compliance with the requirements for new construction.
- The use of traffic control devices. This is generally an inexpensive method of alleviating certain highway defects.
- Median opening modifications.
- Addition of median, channelized islands, and mid-block pedestrian crossings.
- Auxiliary lanes.
- Existing bridges that fail to meet current design standards which are available to bicycle traffic, should be retrofitted on an interim basis as follows: As a general practice, bridges 125 feet in length or longer, bridges with unusual sight problem, steep gradients (which require the cyclist longer time to clear the span) or other unusual conditions should display the standard W11-1 caution sign with an added sign "On Bridge" at either end of the structure. Special care should be given to the right most portion of the roadway, where bicyclists are expected to travel, assuring smoothness, pavement uniformity, and freedom

from longitudinal joints, and to ensure cleanliness. Failure to do so forces bicyclists farther into the center portion of the bridge, reducing traffic flow and safety.

- Addition of bicycle facilities.
- Addition of transit facilities, sidewalks, crosswalks, and other pedestrian features.

3.3.12 Design Exceptions

See **Chapter 14 - Design Exceptions and Variations** for the process to use when the standard criteria found in this Manual cannot be met.

3.3.13 Very Low-Volume Local Roads (ADT ≤ 400)

Where criteria is not specifically provided in this section, the design guidelines presented in Chapter 4 of the ***AASHTO Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400), 1st Edition (2001)*** may be used in lieu of the policies in Chapter 5 of the AASHTO Policy on Geometric Design of Highways and Streets. See **Table 3 – 21 Minimum Lane Widths** for lane widths for very low volume roads.

3.3.13.1 Bridge Width

Bridges are considered functionally obsolete when the combination of ADT and bridge width is used in the National Bridge Inventory Item 68 for Deck Geometry to give a rating of 3 or less. To accommodate future traffic and prevent new bridges from being classified as functionally obsolete, the minimum roadway width for new two lane bridges on very low-volume roads with 20 year ADT between 100 and 400 vehicles/day shall be a minimum of 22 feet. If the entire roadway width (traveled way plus shoulders) is paved to a width greater than 22 feet, the bridge width should be equal to the total roadway width. If significant ADT increases are projected beyond twenty years, a bridge width of 28 feet should be considered. One-lane bridges may be provided on single-lane roads and on two-lane roads with ADT less than 100 vehicles/day where a one-lane bridge can operate effectively. The roadway width of a one-lane bridge shall be 15 ft. One-lane bridges should have pull-offs visible from opposite ends of the bridge where drivers can wait for traffic on the bridge to clear.

3.3.13.2 Roadside Design

Bridge traffic barriers on very low-volume roads must have been successfully crash tested to a Test Level 2 (minimum) in accordance with ***NCHRP Report 350*** or ***Manual for Assessing Safety Hardware (MASH)***.