AGENDA (Draft)

FLORIDA GREENBOOK ADVISORY COMMITTEE MEETING

Thursday, July 19, 2018, 3:00 – 5:00 PM

Haydon Burns Building, Room 314
605 Suwannee Street
Tallahassee, FL 32399-0450

Thursday, July 19, 2018

3:00 – 3:15	**Introductions and General Information**
3:15 – 4:45	**Review of Proposed Revisions for 2018 Greenbook**
	(Geometric Design, Roadside Design, Lighting, Rail-Highway Crossings)
4:45 – 5:00	**Closing Remarks**

CHAPTER 3

GEOMETRIC DESIGN

TABLES

FIGURES

CHAPTER 3

GEOMETRIC DESIGN

A 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*** 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 ~~is~~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)***, also known as the ***AASHTO Greenbook (2011)*** 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 ~~of~~ 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

B 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.

C DESIGN ELEMENTS

C.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.

~~***AASHTO's A Policy on Geometric Design of Highways and Streets (2011)*** may be referenced for a more thorough discussion of design speed.~~

~~Recommended~~ Minimum and maximum values for design speed are given~~provided~~ in Table 3 – 1 Minimum and Maximum ~~Recommended~~ Design Speed. ~~These values should be considered as general guidelines only.~~

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 Greenbook (2011)*** provides additional information on design speed.

Table 3 – 1 Minimum and Maximum ~~Recommended~~ 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	3~~4~~0 – 5~~6~~0 ~~(30 mph min for AADT < 250)~~
			Rolling	2~~3~~0 – 4~~6~~0 ~~(20 mph min for AADT < 50)~~
	Urban	All	All	20 – 30[4]

Footnotes:

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.

C.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 IN 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+45.4=67	4	2.5	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, Design Controls and Criteria, Table 2-1b.

* 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 ~~only limited~~ 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 10mph.

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.***

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
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, Design Controls and Criteria, Table 2-2b.

* 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.

C.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.

C.3.a 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~~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 ~~two feet (~~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 – 3 Minimum Stopping Sight Distance for level and rolling roadways.

Table 3 – 4~~3~~ Minimum Stopping Sight Distance~~s~~

~~MINIMUM STOPPING SIGHT DISTANCES (feet)~~ ~~(For application of stopping sight distance, use an eye height of 3.50 feet and an object height of 2 feet above the road surface)~~											
~~Design Speed (mph)~~	~~20~~	~~25~~	~~30~~	~~35~~	~~40~~	~~45~~	~~50~~	~~55~~	~~60~~	~~65~~	~~70~~
~~Stopping Sight Distance (feet)~~	~~115~~	~~155~~	~~200~~	~~250~~	~~305~~	~~360~~	~~425~~	~~495~~	~~570~~	~~645~~	~~730~~

~~Source: 2011 AASHTO Greenbook, Table 3-1 Stopping Sight Distance on Level Roadways.~~

Design Speed (mph)	Stopping Sight Distance (feet)						
	Level (≤ 2%)	Downgrades			Upgrades		
		3%	6%	9%	3%	6%	9%
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, Table 3-1 Stopping Sight Distance on Level Roadways and Table 3-2 Stopping Sight Distance on Grades.

~~Table 3 – 4~~ **Minimum** ~~Passing~~ **~~Sight Distances~~**

~~MINIMUM PASSING SIGHT DISTANCES (feet)~~ ~~(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)~~											
~~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, Table 3-4 Passing Sight Distance for Design of Two-Lane Highways.~~

C.3.b~~c~~ Decision Sight Distance ~~at Decision Points~~

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. ~~It is desirable to provide sight distances exceeding the minimum at changes in geometry, approaches to intersections, entrances and exits, and other potential decision points or hazards. The sight distance should be adequate to allow the driver sufficient time to observe the upcoming situation, make the proper decision, and take the appropriate action in a normal manner.~~

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~~. In many cases, rapid stopping or lane changing may be extremely undesirable and cause hazardous maneuvers (i.e., in heavy traffic conditions); therefore, it would be preferable to provide sufficient sight distance to allow for a more gradual reaction.~~

Examples of critical locations where additional sight distance is needed include interchange and intersections 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 – 4 Decision Sight Distance may be used (1) to provide values for sight distances that may be appropriate at critical locations, and (2) 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: 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.

~~C.3.d Intersection Sight Distance~~

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

C.3.c~~b~~ 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 – 5~~4~~ Minimum Passing Sight Distances.

Table 3 – 6~~4~~ Minimum Passing Sight Distance

(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)											
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, Table 3-4 Passing Sight Distance for Design of Two-Lane Highways.

C.3.d Intersection Sight Distance

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

C.4 Horizontal Alignment

C.4.a 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 an unexpected and hazardous situation.

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.

C.4.b Maximum Deflections in Alignment without Curves

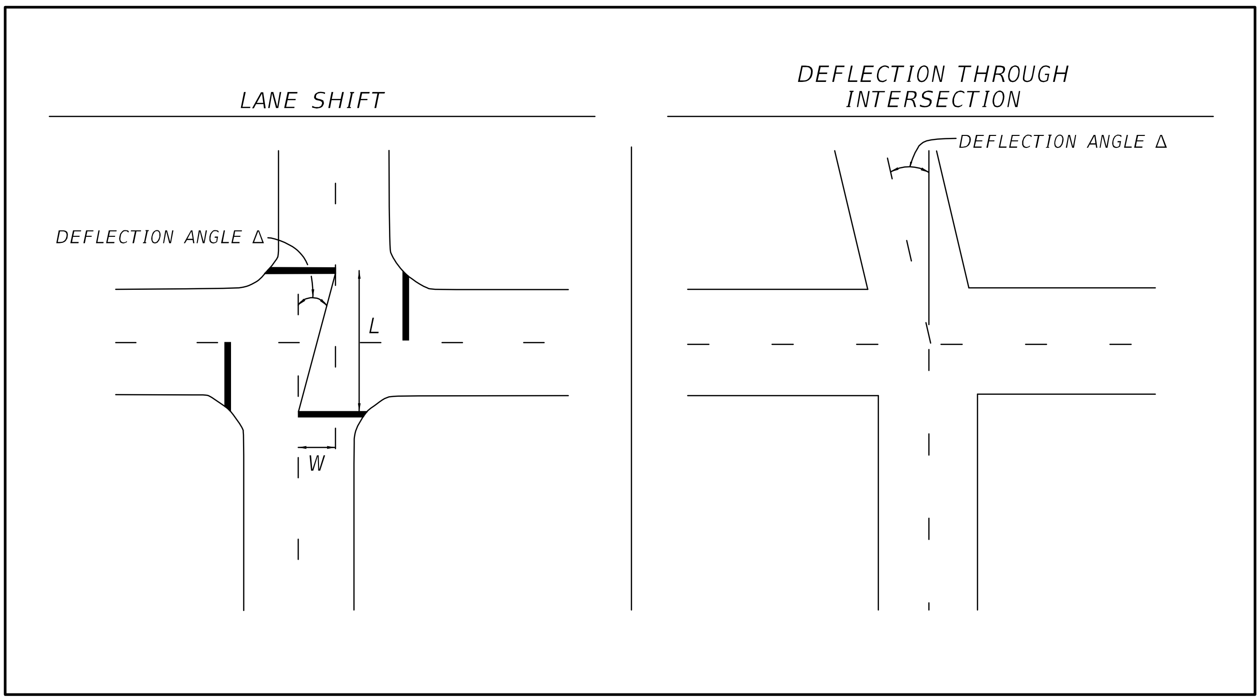
The point where tangents intersect is known as the point of intersection (PI). Although the use of a PI with no horizontal curve is discouraged, there may be conditions where it is necessary. The maximum deflection without a horizontal curve are as follows:

- Flush shoulder and curbed roadways with design speed 40 mph and less is 2o00’00”.
- Flush shoulder roadways with design speed 45 mph and greater is 0o45’00”.
- Curbed roadways with design speed 45 mph and greater is 1o00’00”.
- High speed curbed roadways with design speed 50 mph and greater is 0o45’00”.

Although deflections thru 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 – 6 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.

~~For small deflection angles, curves should be suitably lengthened to avoid the distracting appearance of a kink. Curves should be at least 900 feet long for a central angle of 1 degree or 500 feet long for a central angle of 5 degrees. Gently flowing alignment is generally more pleasing in appearance, as well as, superior from a safety standpoint.~~

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 3-7 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)	5º	4º	3º	2º	1º
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 ft. (Rural) or 100 ft. (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 – 8 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	70	100	120	150	180	200
Minimum Arc Length (feet)	40	50	65	85	100	120	150

C.4.c~~b~~ 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 Department's FDOT Design Manual, and ***A Policy on Geometric Design of Highways and Streets (AASHTO, 2011)***, may be considered.

C.4.c.1 Rural Highways, Urban Freeways and High Speed Urban Highways

The superelevation rates for high speed (50 mph or greater) roadways ~~rural highways, urban freeways, and high speed urban highways~~ are ~~shown~~ provided in Table 3 – 9 Superelevation Rates for Rural Highways, Urban Freeways and High Speed Urban

Highways (e max =0.10)~~Figure 3 – 1 Rural Highways, Urban Freeways and High Speed Urban Highways and Table 35 ~~***.*** These rates are based on Method 5 from the ***2011 AASHTO Greenbook*** using a maximum rate of 0.10 foot per foot of roadway width. ~~Additional superelevation details, given in the Department's Design Standards, may be considered.~~Table 3 – 9~~5~~ also provides the minimum radius required for normal crown without superelevation.

~~Additional superelevation details, given in the Department's Design , may be considered.~~

C.4.c.2 Low Speed Urban Roadways

~~Although superelevation is advantageous for traffic operations~~For low speed (45 mph and less) roadways in urban areas, various factors combine to make ~~its use~~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:

~~The superelevation rates recommended for urban highways and high speed urban streets are shown in Figure 3 – 2 Superelevation Rates (e) For Urban Highways and High Speed Urban Streets. These rates are based on a maximum superelevation rate of 0.05 foot per foot and are recommended for arterials and collectors in built up areas. Additional information regarding superelevation, given in the Department's Design Standards, and **_A Policy on Geometric Design of Highways and Streets (AASHTO, 2011)_**, may be considered.~~**Low Speed Urban Arterials and Collectors**: Superelevation rates for low speed urban arterials and collectors are provided in Table 3 – 10~~6~~ Superelevation Rates for Low Speed Arterials and Collectors (emax = 0.05). These rates are based on the Department's superelevation criteria for low speed arterials and collectors. Table 3 ~~-~~– 10~~6~~ also provides the minimum radius required normal crown without superelevation.

Low Speed Local Roads: Minimum radii for design superelevation rates for low speed local roads are provided in Table 3 ~~-~~– 11~~7~~ Minimum Radii (feet) for Design Superelevation Rates, Low Speed Local Roads (emax = 0.05). These rates are based on Method 2 from the 2011 AASHTO Greenbook. Table 3 ~~-~~– 11~~7~~ also provides the minimum radius required for normal crown (-0.02 ft/ft) without superelevation.

~~Although superelevation is advantageous for traffic operations, various factors combine to make its use 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~~

~~Therefore, horizontal curves on lower speed streets in residential and urban areas are usually designed without superelevation, only side friction being used to counteract the centrifugal force. Figure 3 – 3 Maximum Safe Speed for Horizontal Curves Urban-Lower Speed Streets may be used for determination of the maximum safe speed for horizontal curves on lower speed urban streets.~~

~~Additional information regarding superelevation, given in the Department's Design Standards, and~~ ~~***A Policy on Geometric Design of Highways and Streets (AASHTO, 2011)***~~~~, may be considered.~~

C.4.d~~c~~ 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 ~~relationships are given in Table 3 – 5 Horizontal Curvature~~or minimum radius for the maximum superelevation rates are provided in Tables 3 ~~-~~– 9~~5~~, 3 ~~-~~– 10~~6~~ and 3 ~~-~~– 11~~7~~. The use of sharper curvature ~~for the design speeds shown in Table 3 – 5~~ 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.

~~Figure 3 – 1 Rural Highways, Urban Freeways and High Speed Urban Highways~~

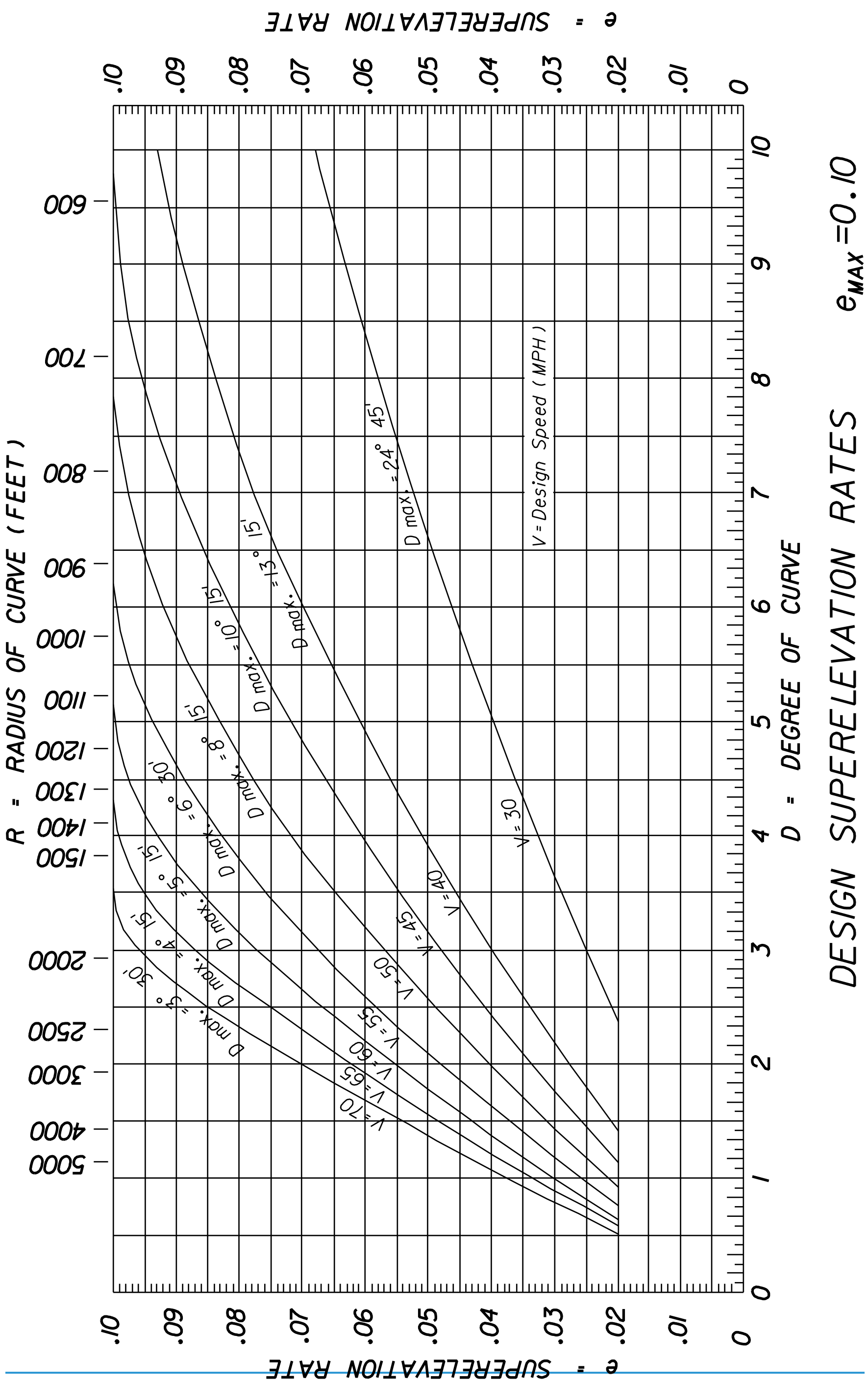
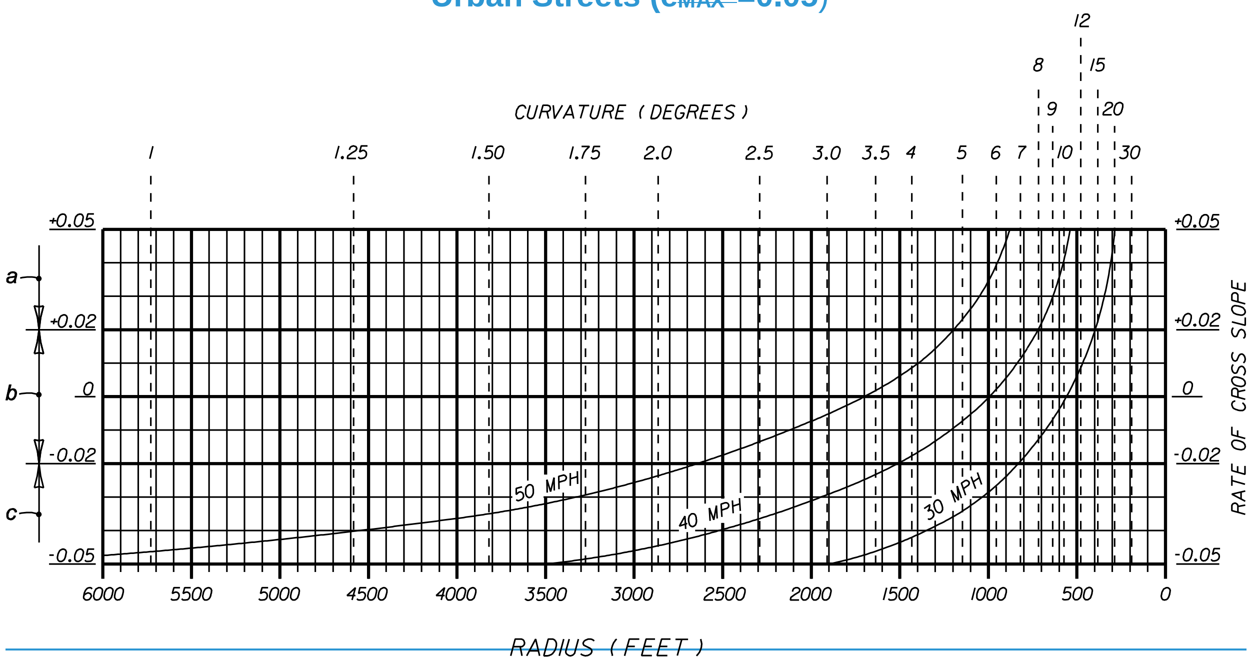


Figure ~~3 – 2 Superelevation Rates (e) For Urban Highways and High Speed Urban Streets (e_{MAX} =0.05~~)



~~a. When the speed curves and the degree of curve lines intersect above this line, the pavement is to be superelevated (positive slope) at the rates indicated at the lines intersecting points.~~

~~b. When the speed curves and the degree of curve lines intersect between these limits, the pavement is to be superelevated at the rate of 0.02 (positive slope).~~

~~c. When the speed curves and the degree of curve lines intersect below this line, the pavement is to have normal crown (typically 0.02 and 0.03 downward slopes).~~

Figure ~~3 – 3 Maximum Safe Speed For Horizontal Curves Urban-Lower Speed Streets~~

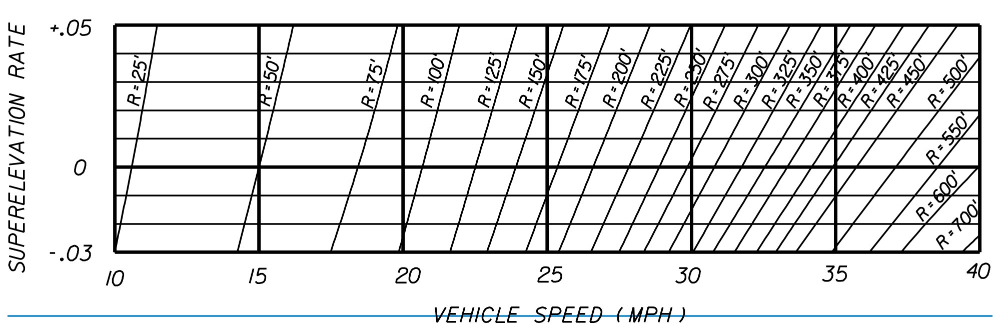


Table 3 ~~-~~– 10~~5~~ Superelevation Rates for Rural Highways, Urban Freeways and High Speed Urban Highways (e_{max} = 0.10)

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'								

* NC/RC and RC/e Break Points (Radius in feet)									
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)

R_{NC}= Minimum Radius for NC R_{RC} = Minimum Radius for RC

Rates for intermediate D and R's are to be interpolated.

Table 3 – 11~~6~~ Superelevation Rates for Low Speed Arterials and Collectors (e_{max} = 0.05)

Degree of Curve D	Radius R (ft.)	Tabulated Values — Design Speed (mph) 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~~7~~ 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
-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

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

~~Figure 3 – 4 Sight Distance on Curves~~

~~C.4.c Curvature~~

~~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 relationships are given in Table 3 – 5 Horizontal Curvature. The use of sharper curvature for the design speeds shown in Table 3 – 5 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 is a significant value in alignment design.~~

~~In urban areas, the density of adjacent development or possibility of congestion act to restrict speeds.~~

~~Table 3 – 5 Horizontal Curvature~~

~~RURAL~~ ~~Based on e_{MAX} = 0.10~~			~~URBAN~~ Arterials and Collectors ~~High-Speed Highways and Streets~~ ~~Based on e_{MAX} = 0.05~~		
~~Design Speed (mph)~~	~~Max. Degree of Curvature~~	~~Min. Radius (feet)~~	~~Design Speed (mph)~~	~~Max. Degree of Curvature~~	~~Min. Radius (feet)~~
~~20~~	~~79° 30'~~	~~75~~	~~---~~	~~---~~	~~---~~
~~25~~	~~45° 15'~~	~~130~~	~~---~~	~~---~~	~~---~~
~~30~~	~~28° 30'~~	~~200~~	~~30~~	~~23° 45'~~	~~245~~
~~35~~	~~19° 30'~~	~~295~~	~~35~~	~~16° 00'~~	~~360~~
~~40~~	~~13° 45'~~	~~415~~	~~40~~	~~11° 15'~~	~~510~~
~~45~~	~~10° 30'~~	~~540~~	~~45~~	~~8° 15'~~	~~680~~
~~50~~	~~8° 15'~~	~~695~~	~~50~~	~~6° 30'~~	~~880~~
~~55~~	~~6° 30'~~	~~880~~	~~55~~	~~5° 00'~~	~~1125~~
~~60~~	~~5° 15'~~	~~1095~~	~~---~~	~~---~~	~~---~~
~~65~~	~~4° 15'~~	~~1345~~	~~---~~	~~---~~	~~---~~
~~70~~	~~3° 30'~~	~~1640~~	~~---~~	~~---~~	~~---~~

~~LOW-SPEED URBAN STREETS~~ Local				
~~Design Speed (mph)~~	~~With e_{MAX} = 0.05~~		~~Without Superelevation (e_{MAX} = -0.02)~~	
	~~Max. Degree of Curvature~~	~~Min. Radius (feet)~~	~~Max. Degree of Curvature~~	~~Min. Radius (feet)~~
~~20~~	~~68° 45'~~	~~85~~	~~53° 30'~~	~~110~~
~~25~~	~~38° 30'~~	~~150~~	~~28° 45'~~	~~200~~
~~30~~	~~23° 45'~~	~~240~~	~~17° 00'~~	~~335~~

~~(TABLE CONTINUES ON NEXT PAGE)~~

~~Table 3 – 5 Horizontal Curvature~~
~~(Continued)~~

~~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~~	~~79° 30'~~	~~15~~
~~25~~	~~45° 15'~~	~~17~~
~~30~~	~~28° 30'~~	~~18~~
~~35~~	~~19° 30'~~	~~20~~
~~40~~	~~13° 45'~~	~~22~~
~~45~~	~~10° 30'~~	~~24~~
~~50~~	~~8° 15'~~	~~27~~
~~55~~	~~6° 30'~~	~~29~~
~~60~~	~~5° 15'~~	~~31~~
~~65~~	~~4° 15'~~	~~33~~
~~70~~	~~3° 30'~~	~~35~~

C.4.e~~d~~ Superelevation Transition (superelevation runoffs plus tangent runoff)

Superelevation runoff is the general term denoting the length of street or highway needed to ~~accomplish~~ 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. ~~Spiral curves can 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.~~

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~~ and gutter~~).

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 ~~-~~–12 Superelevation Transition Slope Rates. The 2011 AASHTO Greenbook provides ~~may be referenced for~~ additional information on superelevation transition design.

The Department's ~~Design Standards~~ ***Standard Plans for Road and Bridge Construction*** ~~show in~~ provide additional information on ~~detail~~ superelevation transitions for various sections and methods for determining length of transition.

Table 3 – 13 Superelevation Transition Slope Rates

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

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 ***Standard Plans, Index 000-511***.

Spiral curves may~~can~~ 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 ***2011 AASHTO Greenbook***.

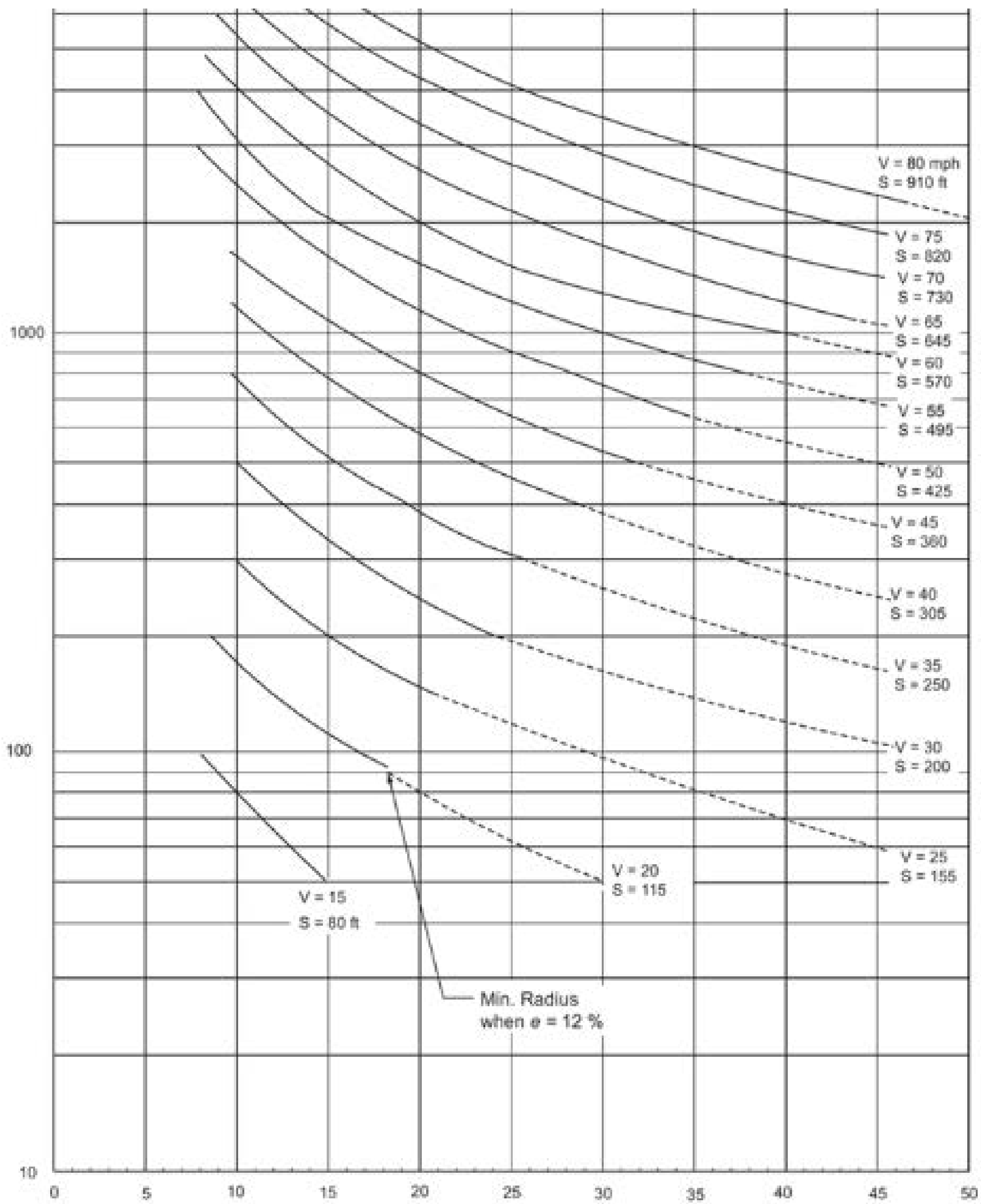
~~The Department's Design Standards show in detail superelevation transitions for various sections and methods for determining length of transition.~~

C.4.f 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 Design Controls for Stopping Sight Distance on Horizontal Curves (emax ≤ 0.02) shows the horizontal sight line offsets needed for clear sight areas that satisfy stopping sight distance criteria presented in Table 3 – 3 Minimum Stopping Sight Distances for horizontal curves of radii on flat grades.

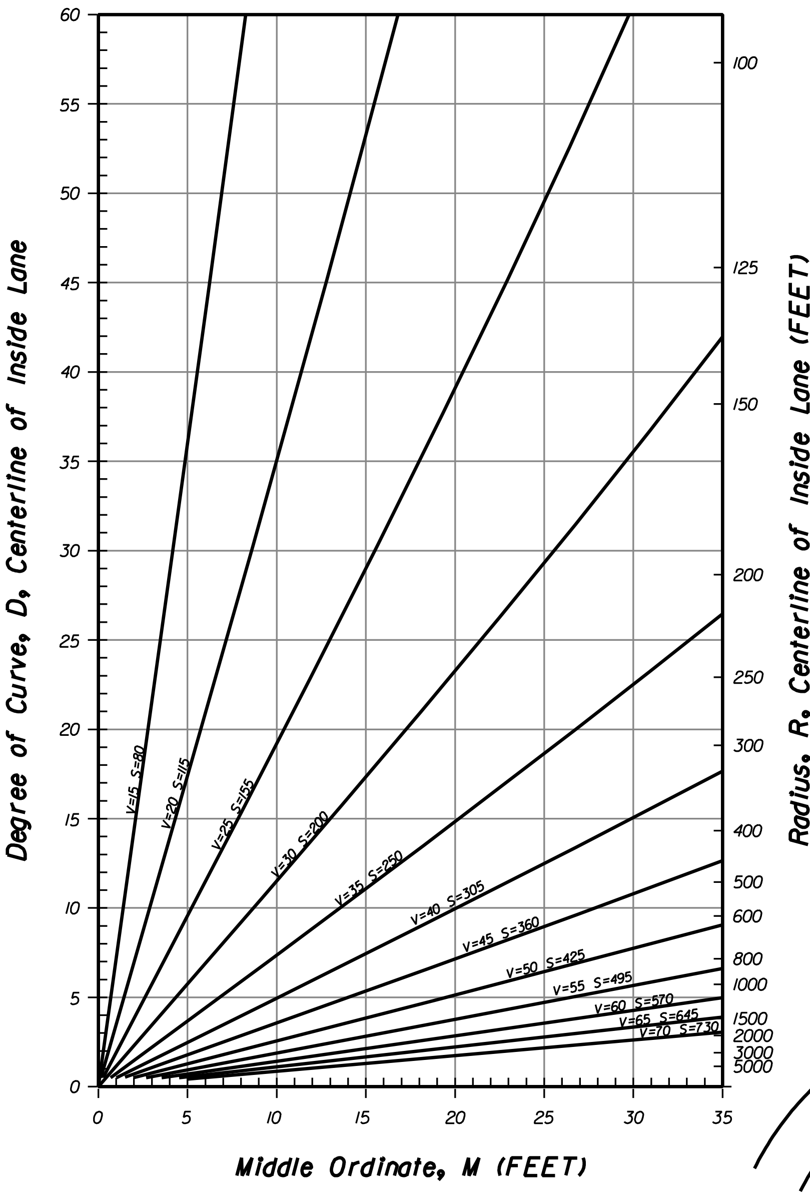
Figure 3 – 14 Design Controls for Stopping Sight Distance on Horizontal Curves (emax ≤ 0.02)

Radius, R, Centerline of Inside Lane (feet)



Horizontal Sight Line Offset, HSO, Centerline Inside Lane to Obstruction (feet)

Source: 2011 AASHTO Greenbook, Figure 3 – 22b. Design Controls for Stopping Sight Distance on Horizontal Curves.



$$M = \frac{5730}{D}\left(1 - \mathrm{Cos}\frac{SD}{200}\right)$$

$$R = \frac{5730}{D} \quad \text{and} \quad \theta = \frac{SD}{200}$$

$$M = R\,(1 - \mathrm{Cos}\,\theta)$$

$$M = R\left(1 - \mathrm{Cos}\,\frac{28.65\,S}{R}\right)$$

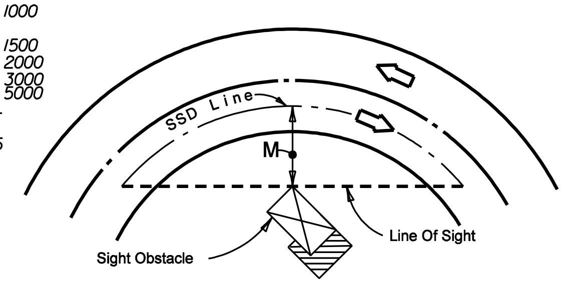
Where

S = Stopping Sight Distance (FEET)

D = Degree of Curve

M = Middle Ordinate (FEET)

R = Radius (FEET)



RELATION BETWEEN DEGREE OF CURVE AND VALUE OF MIDDLE ORDINATE NECESSARY TO PROVIDE STOPPING SIGHT DISTANCE ON HORIZONTAL CURVES UNDER OPEN ROAD CONDITIONS.

Table 3 – 14~~5~~ Horizontal Curvature ~~(Continued)~~

Lateral Clearance From Edge Of Traveled Way To Obstruction For Maximum Curvature (Degrees), Based On Line Of Sight On Inside Lane (Lateral Clearance = $M_{\text{Inside Lane}}$ – 6') Based on e_{MAX} = 0.10		
Design Speed (mph)	Maximum Curvature	Clearance (feet)
20	57~~79~~° 45~~30~~'	11~~5~~
25	36~~45~~° 15'	13~~7~~
30	24~~28~~° 45~~30~~'	16~~8~~
35	17~~19~~° 45~~30~~'	19~~20~~
40	13° 30~~45~~'	21~~2~~
45	10° 15~~30~~'	23~~4~~
50	8° 15'	27
55	6° 30'	29
60	5° 15'	31
65	4° 15'	33
70	3° 30'	35

C.4.g~~e~~ 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 – 14~~6~~A Calculated and Design Values for Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way and 3 – 14~~6~~B 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.

Table 3 – 15~~6~~A 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, Table 3 – 26b ~~Calcualted~~Calculated and Design values for Traveled Way Widening on Open Highway Curves.

Notes: 1. Values shown are for WB-62~~50~~ design vehicle and represent widening in feet. For other design vehicles, use adjustments in Table 3-14~~6~~B.

Table 3 – 15~~6~~B
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-62~~	~~WB-65~~	WB-627FLD	WB-67100T	WB-67109D
7000	-1.12	-1.21	0.1	0.1	0.0	0.10	-0.13
6500	-1.31	-1.21	0.1	0.1	0.0	0.1	-0.13
6000	-1.32	-1.21	0.1	0.2	0.0	0.1	-0.23
5500	-1.32	-1.21	0.1	0.2	0.0	0.1	-0.24
5000	-1.32	-1.31	0.1	0.2	0.0	0.1	-0.24
4500	-1.42	-1.31	0.1	0.2	0.0	0.1	-0.25
4000	-1.42	-1.32	0.2	0.2	-0.1	0.1	-0.25
3500	-1.53	-1.42	0.2	0.3	-0.1	0.1	-0.36
3000	-1.63	-1.42	0.2	0.3	-0.1	0.1	-0.37
2500	-1.74	-1.52	0.3	0.4	-0.1	0.21	-0.48
2000	-1.85	-1.63	0.3	0.5	-0.1	0.2	-01.50
1800	-1.95	-1.73	0.4	0.5	-0.1	0.2	-01.51
1600	-21.06	-1.84	0.4	0.6	-0.1	0.2	-01.63
1400	-21.27	-1.94	0.5	0.6	-0.2	0.32	-01.65
1200	-21.48	-21.15	0.5	0.8	-0.2	0.3	-01.87
1000	-2.70	-21.36	0.6	0.9	-0.2	0.43	-02.90
900	-2.81	-21.47	0.7	1.0	-0.2	0.4	-12.03
800	-32.12	-21.68	0.8	1.1	-0.3	0.54	-12.16
700	-32.4	-21.9	0.9	1.3	-0.3	0.65	-12.39
600	-32.86	-32.20	1.1	1.5	-0.4	0.76	-13.54
500	-42.39	-32.62	1.3	1.8	-0.4	0.87	-14.81
450	-43.72	-32.94	1.4	2.0	-0.5	0.97	-24.06
400	-53.24	-42.35	1.6	2.3	-0.5	10.08	-25.31
350	-53.8	-42.78	1.9	2.6	-0.6	1.10	-25.69
300	-64.63	-53.40	2.2	3.0	-0.7	1.31	-36.09
250	-74.79	-63.35	2.6	3.7	-0.9	1.64	-38.63
200	-95.49	-74.61	3.3	4.6	-1.1	21.07	-410.65

Source: 2011 AASHTO Greenbook, Table 3 - 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 3-6A.

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.

C.5 Vertical Alignment

C.5.a 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.

C.5.b 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 – 15~~7~~ Recommended 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 – 2~~5~~ 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 – 16~~7~~ ~~Recommended~~ 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	---	---
~~Industrial**~~		~~---~~	~~---~~	~~4~~	~~4~~	~~4~~	~~4~~	~~3~~	~~3~~	~~3~~	~~---~~	~~---~~	~~---~~	~~---~~	~~5~~	~~5~~	~~5~~	~~5~~	~~4~~	~~4~~	~~4~~	~~---~~	~~---~~

Source: 2011 AASHTO Greenbook, Tables 5–2, 6~~-~~–2~~1~~, 6~~-~~–8, 7~~-~~–2, 7~~-~~–4, 8~~-~~–1.

Notes: 1. Grades 1% steeper than the value shown may be provided in urban areas with ~~where~~ right of way ~~is~~ constraints~~ed~~.

2. Short lengths of grade ~~in rural areas~~ (≤ 500 feet in length), one-way downgrades, and grades on low volume ~~rural~~ collectors may be up to 2% steeper than the grades shown above.

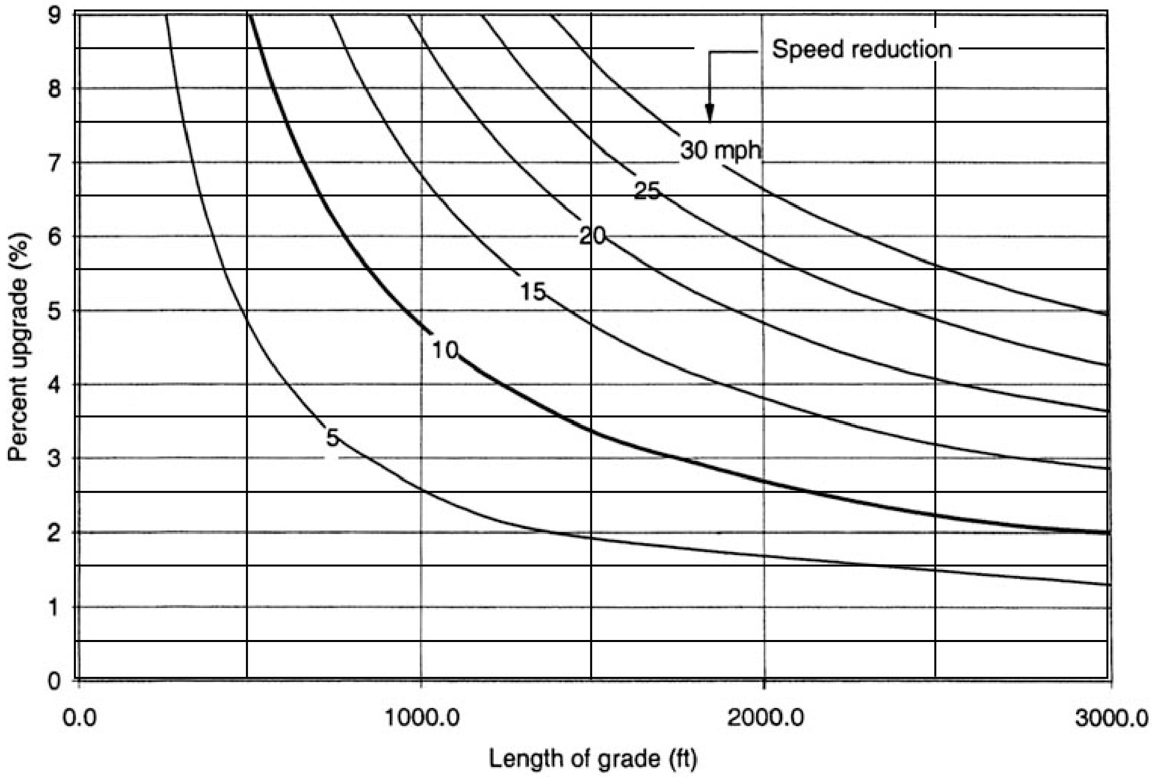
3. Residential street grade should be as level as practical, consistent with surrounding terrain, and less than 15%. Streets in commercial or industrial areas should have grades less than 8%, and flatter grades should be encouraged.

~~* May be increased by 32 percent for urban streets under extreme conditions.~~

~~** Local and collector streets with significant (15% or more) truck traffic.~~

~~For short sections less than 500' and for one-way downgrades, the maximum gradient may be 1% steeper.~~

Figure 3 – 2~~5~~ Critical Length Versus Upgrade



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

Source: 2011 AASHTO Greenbook, ~~A Policy on Geometric Design of Highways and Streets,~~ Figure 3-28.

C.5.c 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 – 16~~8~~ Maximum Change in Grade Without Using Vertical Curve. Table 3 – 17~~9~~ Rounded K Values for Minimum Lengths Vertical Curves provides additional information.

~~–~~The length of vertical curve on a crest, as governed by stopping sight distance, is obtained from Figure 3 – 3~~6~~ Length of Crest Vertical Curve (Stopping Sight Distance). The minimum length of a crest vertical curve to obtain minimum passing sight distance is given in Figure 3 – 4~~7~~ Length of Crest Vertical Curve (Passing Sight Distance). The minimum length of a sag vertical curve, as governed by vehicle headlight capabilities, is obtained from Figure 3 - 5~~8~~ 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 – 17~~8~~ 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 – 18~~9~~ Rounded K Values for Minimum Lengths Vertical Curves

~~Rounded K Values For Minimum Lengths Vertical Curves~~

(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

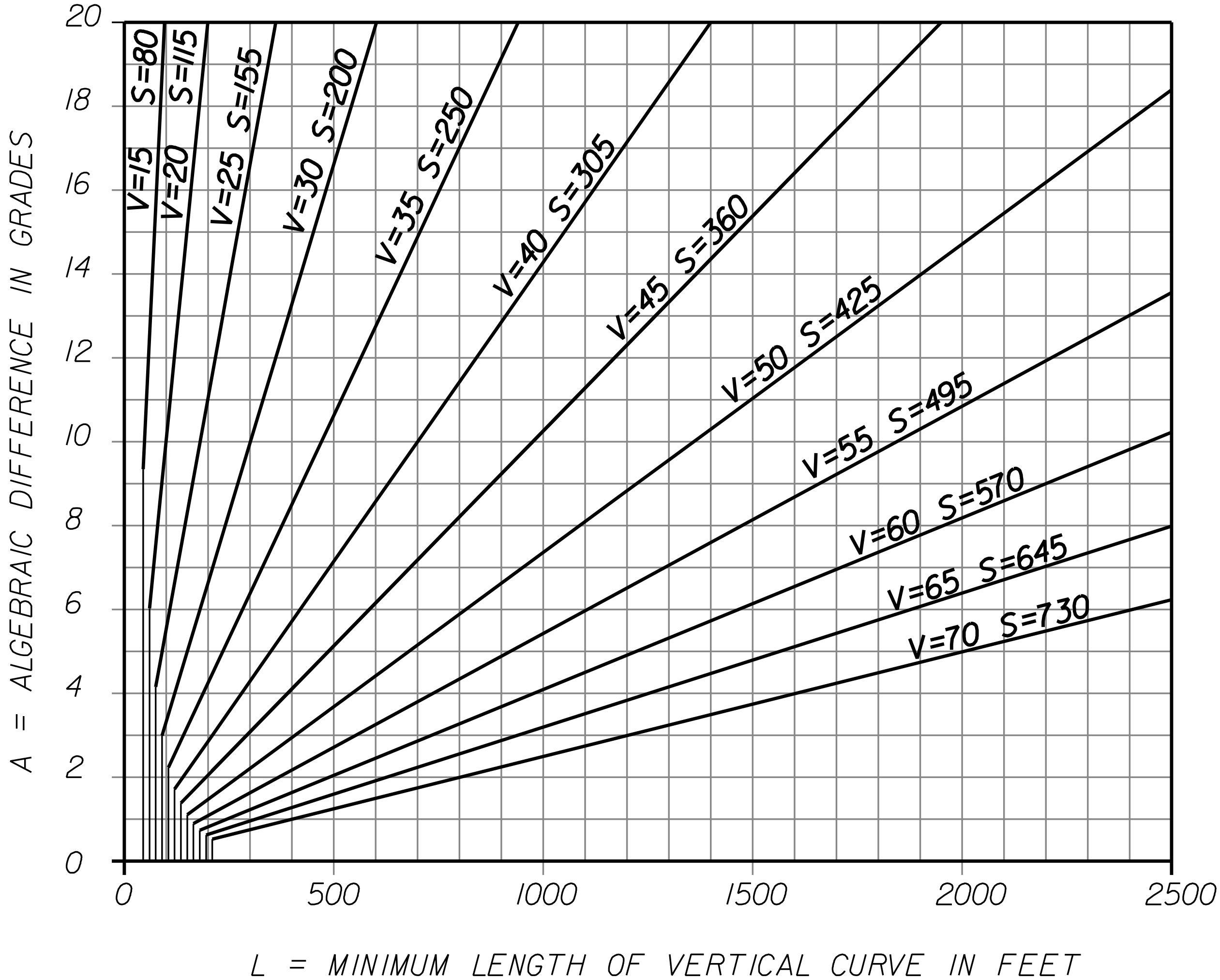
Design Speed (mph)		20	25	30	35	40	45	50	55	60	65	70
K Values for Crest Vertical Curves		7	12	19	29	44	61	84	114	151	193	247
K Values for Sag Vertical Curves		17	26	37	49	64	79	96	115	136	157	181

- 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

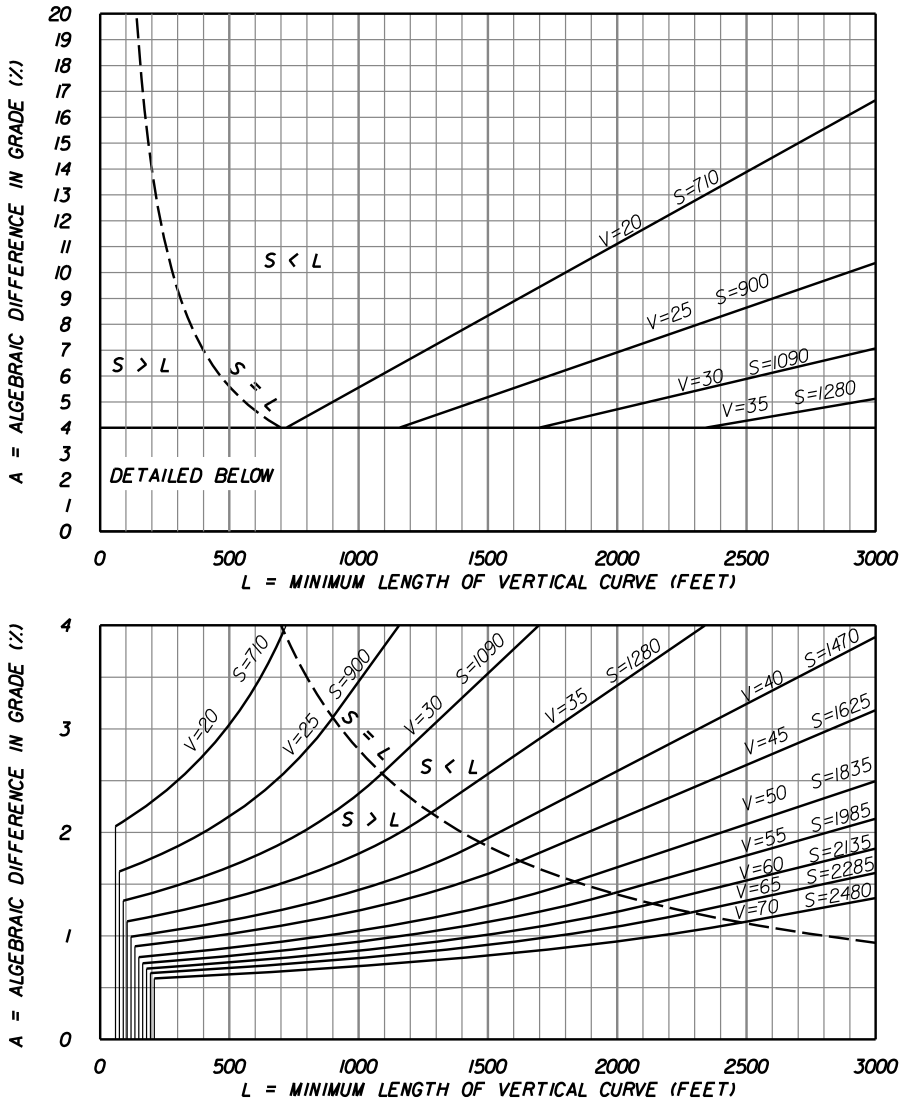
Figure 3 – 3~~6~~ Length of Crest Vertical Curve (Stopping Sight Distance)



Lengths of vertical curves are computed from the formula: $L = \frac{AS^2}{1329}$

A = Algebraic Difference In Grades In Percent
S = Sight Distance
L = Minimum Length of Vertical Curve In Feet

Figure 3 – 4~~7~~   Length of Crest Vertical Curve (Passing Sight Distance)



The sight distance is computed from the following formulas:

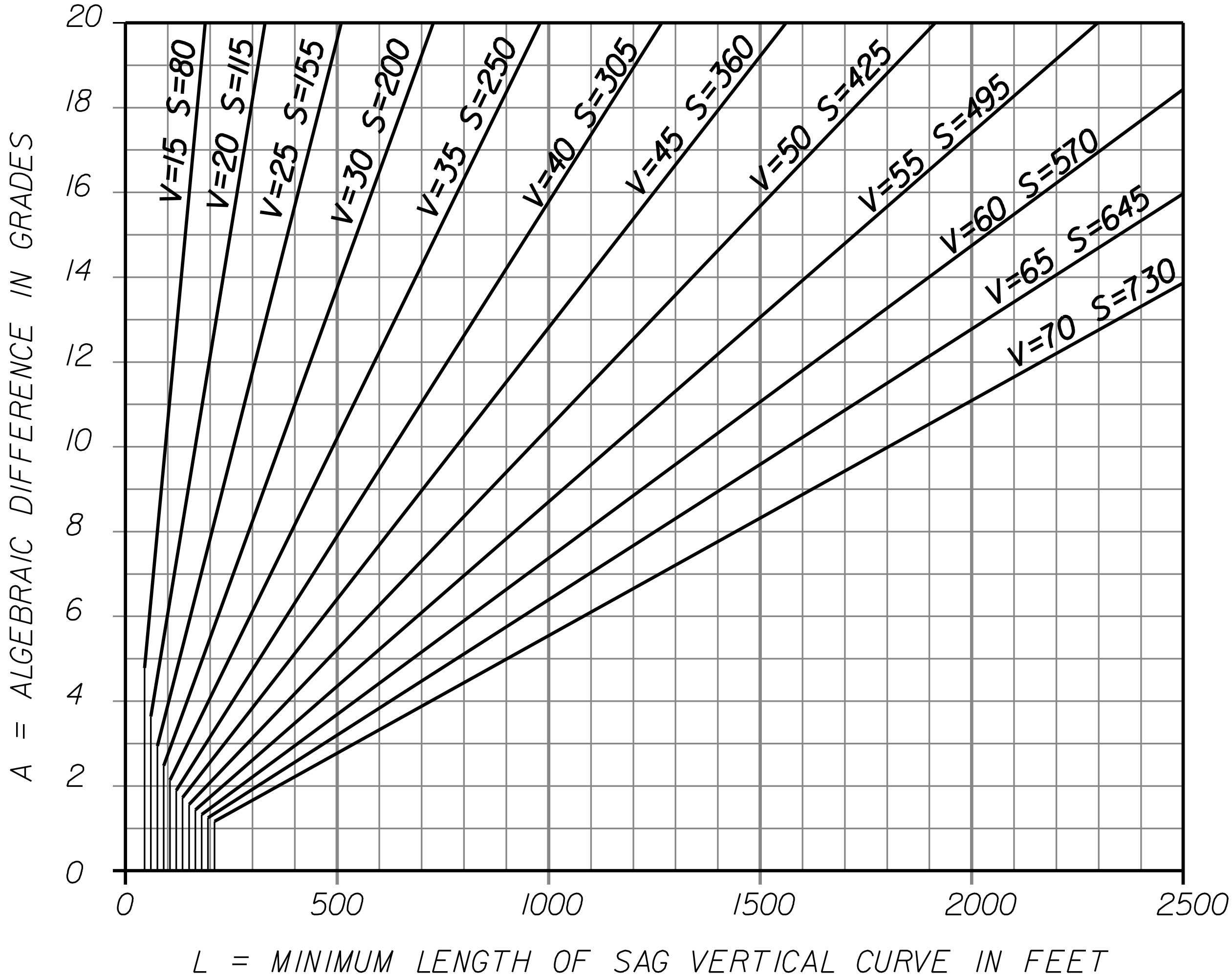
$$S < L,\quad L = \frac{AS^2}{2800} \qquad S > L,\quad L = 2S - \frac{2800}{A}$$

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

Figure 3 – 5~~8~~ Length of Sag Vertical Curve (Headlight Sight Distance)

Lengths of vertical curves are computed from the formula:

$$L = \frac{AS^2}{400 + 3.5(S)}$$



C.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.

C.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.

C.7.a Number of Lanes

The number of travel lanes is determined by several interrelated factors such as capacity, level of service, and service volume. (***A Policy on Geometric Design of Highways and Streets (AASHTO, 2011***), and the ~~current~~ ***Highway Capacity Manual (TRB, 2010***).

C.7.b 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***.

C.7.b.1 Pavement Width

Minimum lane widths for travel lanes, speed change lanes, turn lanes and passing lanes are provided in Table 3 – 17~~0~~ Minimum Lane Widths. The table applies to both divided and undivided facilities. For Information on parking lane dimensions, see Section C.7.h Parking of this Chapter.

On existing multilane ~~urban~~ curbed ~~and gutter~~ 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 – 19~~0~~ Minimum Lane Widths

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

See Footnotes on next page

Footnotes

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~~severely limited~~.

3. In constrained areas where truck ~~and bus~~ volumes are low and speeds are <~~less than~~ 35 mph, 10’ lanes may be used.

~~3.~~4. On roadways with ~~In constrained areas with~~ a transit route, a minimum of 11’ outside lane width is~~s~~ ~~are~~ required.

~~4.~~5. In residential areas where right of way is severely limited, 9’ may be used.

~~5.~~6. Turn lane width in raised or grass m~~M~~edians ~~turn lane widths~~ shall not exceed 14’ ~~15’~~. 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 ~~existing curb locations are fixed~~ 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.

~~6.~~7. Turn Lane width should be same as Travel Lane width. May be reduced to 10’ where right of way is constrained.

~~7.~~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.

~~8.~~10. ~~Table applies to both divided and undivided facitlties~~

C.7.b.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.

C.7.c 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

C.7.c.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. ~~Since the function of the shoulders is to provide an emergency storage or travel path, the desirable width of all shoulders should be at least 10 feet. Where economic or practical constraints are severe, it is permissible, but not desirable, to reduce the shoulder width.~~

~~Outside shoulders shall be provided on all streets and highways with open drainage and should be at least 6 feet wide. Facilities with a heavy traffic volume or a significant volume of truck traffic SHOULD have outside shoulders at least 8 feet wide.~~ The minimum width of outside and median shoulders is provided in Table 3 – 19 Minimum Shoulder Widths for Flush Shoulder Highways. S~~s~~houlders for two-lane, two-way highways~~roadwaysshoulders~~ are based upon traffic volumes~~shall not be less than the values given in Table 3 – 11 Shoulder Widths for Rural Highways~~. 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~~s~~, 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 Department's ***Standard Plans Index 700-106*** provides information on signing and marking the approaching shoulder.

~~Median shoulders are desirable on all multi-lane, non-curb and gutter divided streets and highways. For shoulder widths on multi-lane divided highways see Table 3 – 11.~~

Table 3 – 20~~11~~ Minimum Shoulder Widths for ~~-~~Flush Shoulder ~~Rural~~ Highways

Two Lane Undivided

Design Speed (mph)	Average Daily Traffic (2 – Way)		
	0 - 400	400 - 750	≥750 - ~~1600~~
All	2 feet	6 feet	8 feet

Multilane Divided

Number of Lanes Each Direction	Shoulder Width (feet)			
	Outside		Median	
	Roadway	Bridge	Roadway	Bridge
2	8~~10~~ (min.)	8~~10~~	4~~6~~ (min.)	4~~6~~
3 or more	10 (min.)	10	6~~10~~ (min.)	6~~10~~

C.7.c.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 – 20~~12~~ Shoulder Cross Slope.

Table 3 – 21~~12~~ Shoulder Cross Slope

	Shoulder Type		
	Paved	Gravel or Crushed Rock	Turf
Shoulder Cross Slope (Percent)	2 to 6%	4 to 6%	6 to 8%

Notes: 1. Existing shoulder cross slope (paved and unpaved) ≤ 12% may remain.

Source – 2011 AASHTO Greenbook, 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.

C.7.d Sidewalks

The design of sidewalks is affected by many factors, including~~, but not limited to,~~ traffic characteristics, pedestrian volume, roadway type, ~~characteristics of vehicular traffic~~, and other design elements. ***Chapter 8 - Pedestrian Facilities*** of this Manual and ***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 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 in a rural area should be based on engineering judgment, after observation of existing pedestrian traffic and expectation of additional demand, should a sidewalk be made available.

Sidewalks should be constructed as defined in this Manual. ~~-~~***Chapter 8 - Pedestrian Facilities***, ~~.~~***Chapter 10 – Maintenance and Resurfacing*** and ***Section C.10.a.3 – Sidewalks and Curb Ramps*** of this chapter~~***Chapter 8 - Pedestrian Facilities***~~ 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.

In areas of high use, refer to the ***Highway Capacity Manual, Volume 3, Chapter 23, Off-Street Pedestrian and Bicycle Facilities (2010)*** for calculation of appropriate additional width.

Curb ramps shall be provided at all intersections with curb (***Section 336.045 (3), Florida Statutes***). Each crossing should have separate curb ramps, perpendicular with the curb, and landing within the crosswalk.~~parallel with the direction of travel~~. In addition to the design criteria provided in this chapter, the ***2006 Americans with Disabilities Act Standards for Transportation Facilities*** as required by 49 C.F.R 37.41 or 37.43 and the ***2012 Florida Accessibility Code for Building Construction*** as required by 61G20-4.002 impose additional requirements for the design and construction of pedestrian facilities.

C.7.e 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.

C.7.e.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.

C.7.e.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 – 20~~13~~ Minimum Median Width~~Median Width for Freeways (Urban and Rural)~~. 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 – 20~~14~~ Minimum Median Width ~~Median Width for Urban and Rural Multilane Streets and Highways (Multilane Facilities)~~. On urban streets, the median widths shall not be less than the values given in Table 3 – 20~~14~~. 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 (C.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 ~~auxiliary~~ 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 – 13 Minimum Median Width for Freeways~~
~~(Urban and Rural)~~

~~Design Speed (mph)~~	~~Without Barrier Minimum Permitted Median Width (feet)~~	~~With Barrier (feet)~~
~~60 and Over~~	~~60 1**~~	~~26 2~~
~~Under 60~~	~~40 *~~	~~26 2~~

~~* Applicable for urban areas ONLY.~~

~~Notes:~~

~~**1. Applicable for new construction ONLY.~~
~~(40 feet minimum allowed when lanes added to median).~~
~~2. Based on 2 ft. median barrier and 12 ft. shoulder.~~

~~Table 3 – 14 Minimum Median Width for Streets and Highways~~
~~Rural and Urban~~
~~Streets and Highways~~

~~(Multilane Rural Facilities)~~

~~DesignESIGN SpeedPEED (mph)~~	~~MINIMUM WidthIDTH (feet)~~
~~504555 and Over~~	~~40~~
~~Under 504555~~	~~22 3~~

~~Median Width for (Multilane Urban Streets)~~

~~Design SpeedESIGN SPEED (mph)~~	~~MINIMUM WidthIDTH (feet)~~
~~50~~	~~19.5~~
~~45 and LESS~~	~~15.5~~

~~Paved medians with a minimum width of 10 feet may be used for two-way turn lanes and painted or raised medians when design speeds are 40 mph or less.~~

Table 3 – 22~~13~~ Minimum Median Width

Type of Facility	Width (feet)
Freeways	
~~Interstate, Without Barrier~~	~~64 [1]~~
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~~45~~ mph	40
Design Speed ≤~~<~~ 45 mph	22 [2]
Paved and Painted for Left Turns	See Table 3 – 17 Minimum Lane Widths ~~12~~ [3]

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

Footnotes:

1. Based on 2 ft. wide, concrete median barrier and 12 ft. shoulder.

2. On ~~reconstruction~~ projects where right of way is constrained~~existing curb locations are fixed due to severe right of way constraints~~, 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~~. Restricted to 5-lane sections with design speeds ≤ 4 mph. On reconstruction projects where existing curb locations are fixed due to severe right of way constraints, the minimum width may be reduced to 10 ft. These flush medians are to include sections of raised or restrictive median for pedestrian refuge.~~

C.7.e.3 Median Slopes

A vehicle should be able to tra~~ns~~verse 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 than1:6 (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 %).

C.7.e.4 Median Barriers

~~The primary objective for placing a barrier structure in the median is to prevent vehicles from entering the opposing traffic stream, either accidentally or intentionally. Median barriers may also be used to reduce the glare produced by oncoming vehicle headlights. When selecting the type of barrier, care should be exercised to avoid headlight flicker through barriers.~~

~~The use of median barriers to reduce horizontal separation is permitted on facilities with substantially full control of access. Frequent openings in the barrier for intersections or crossovers expose the barrier end, which constitute severe hazard at locations with an inherently high crash potential and should be shielded. Median barriers may be considered for urban freeways and high speed arterials with controlled access.~~

~~Median barriers shall be used on controlled access facilities if the median width is less than the minimum permitted values given in Table 3 – 13.~~ See ***Chapter 4 – Roadside Design*** for ~~additional~~criteria on ~~for~~ median barriers. The ***AASHTO Roadside Design Guide*** provides additional information and guidelines on the use of median barriers.

~~The median barrier should not be placed closer than 4 feet10 feet from the inside edge of traveled way. Further requirements for median barriers are given in ***Chapter 4 – Roadside Design***.~~

C.7.f 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:

1. Channelization — To control and direct traffic movement, usually turning;

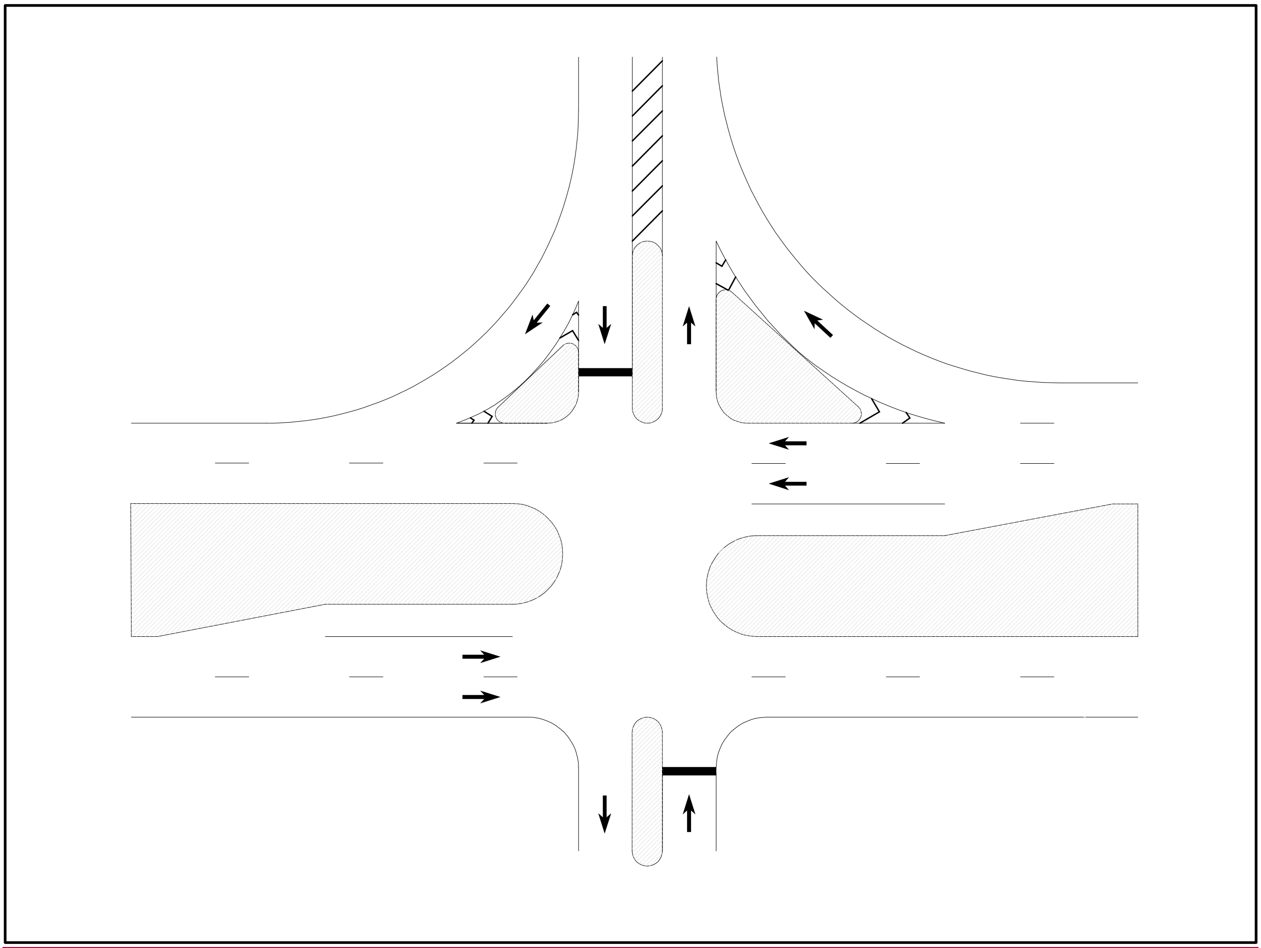
2. Division — To divide opposing or same direction traffic streams, usually through movements; and

3. 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 ~~particular intersection~~intersection design as illustrated in Figure 3 – 6~~9~~ General Types and Shapes of Islands and Medians. They should conform to the general principles that follow.

Figure 3 – 6~~9~~ 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 ***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 that the ground cover is cost-effective and can be properly maintained. ~~***Index 546*** of~~ T~~t~~he Department's ***Design Manual, Chapter 212 Intersections*** ~~***Design Standards***~~ 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 ***AASHTO Roadside Design Guide.***

C.7.f.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~~3~~ – 2~~16~~ Lateral Offset shall be provided. Mast arm ~~bases and~~ foundation diameters ~~shafts~~vary~~in width, ranging~~ from 3.5 feet to ~~4.5~~ 5.0 feet~~in diameter~~. 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 3-8~~10~~ and 3-9~~11~~ Details of Corner Island for Turning Roadways (Curbed~~ and Gutter~~) and (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 ***~~Index 17346~~*** ~~of~~ the Department's ***Standard Plans, Index 711-001***~~***Design Standards***~~.

Figure 3 – 7~~xx~~ Channelization Island for Pedestrian Crossings (Curbed~~ and Gutter~~)

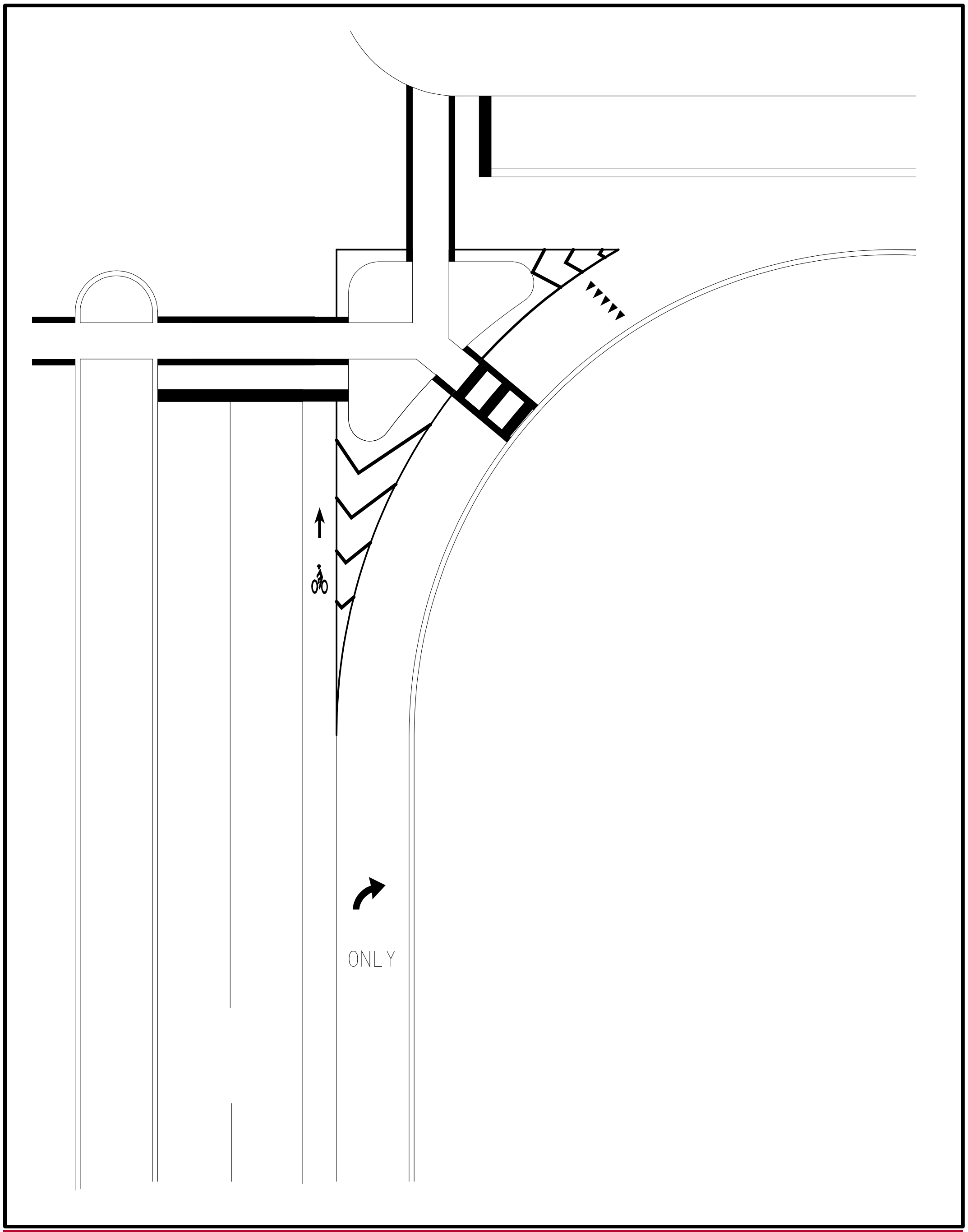
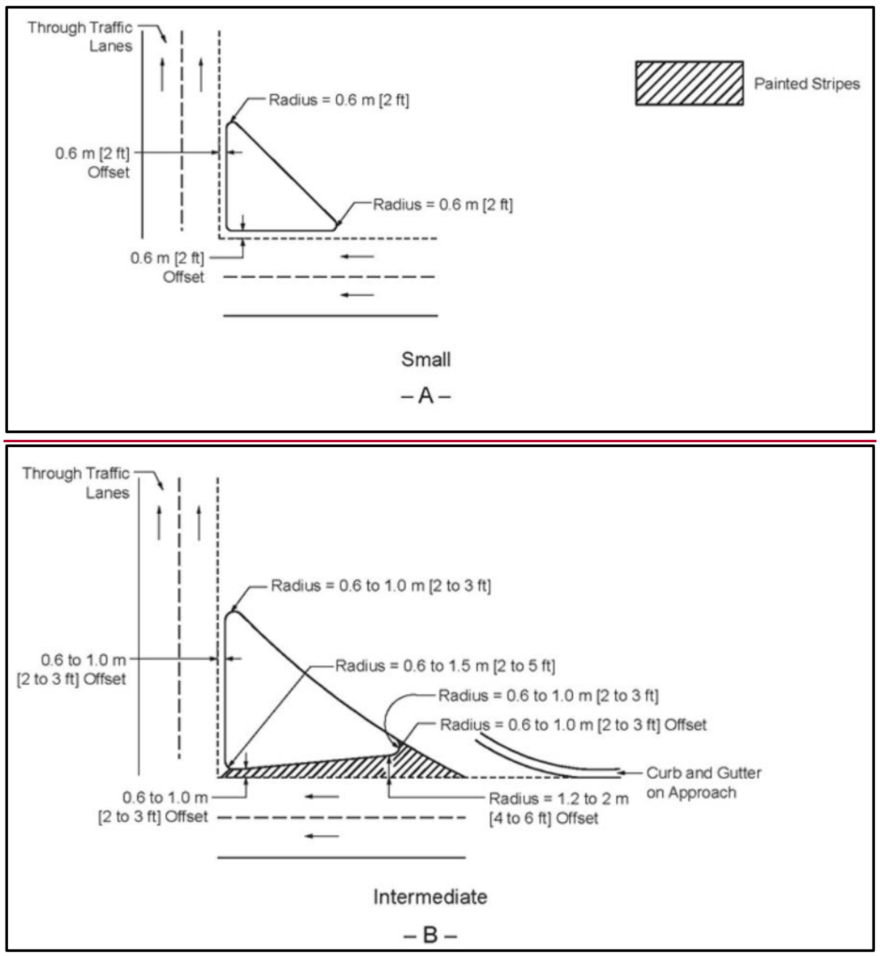
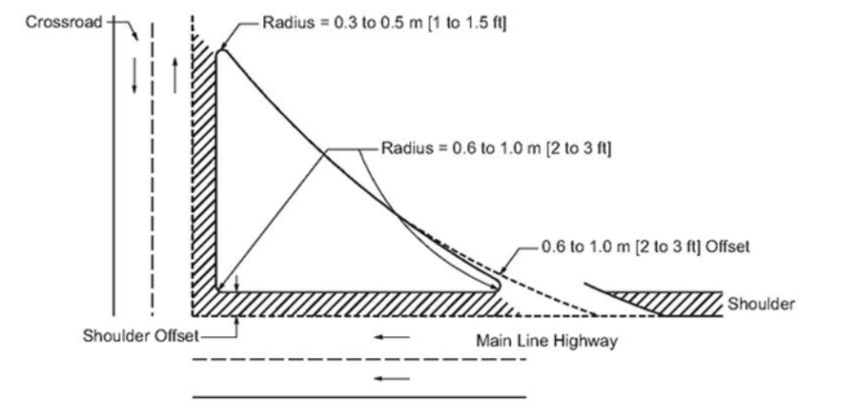


Figure 3 – 8~~10~~ Details of Corner Island for Turning Roadways (Curbed~~ and Gutter~~)

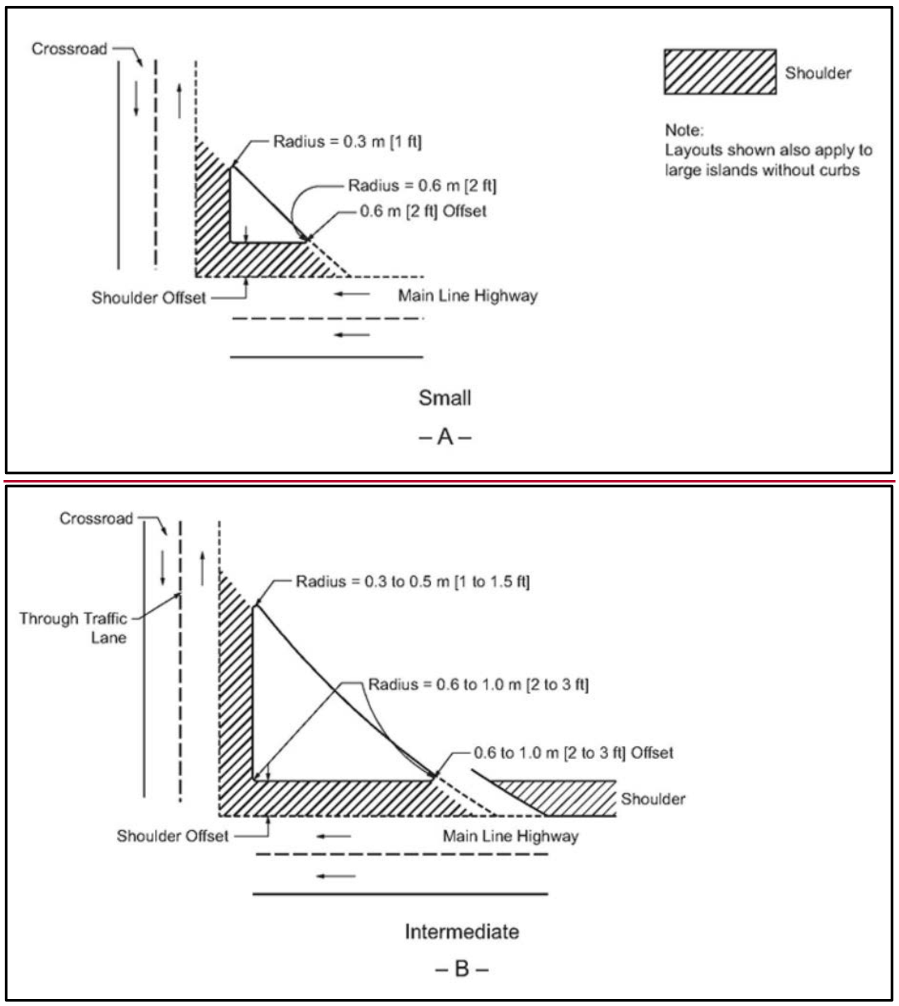


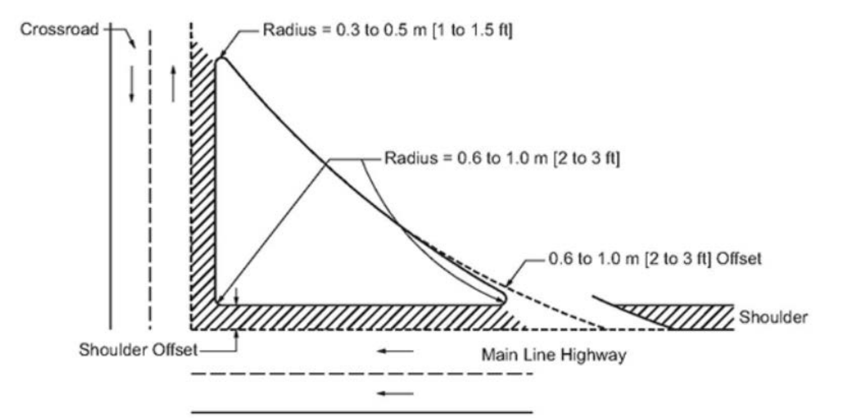

Crossroad
Radius = 0.3 to 0.5 m [1 to 1.5 ft]
Radius = 0.6 to 1.0 m [2 to 3 ft]
0.6 to 1.0 m [2 to 3 ft] Offset
Shoulder
Shoulder Offset
Main Line Highway

Large

– C –

Figure 3 – 9~~11~~ Details of Corner Island for Turning Roadways (Flush Shoulder)




Crossroad
Radius = 0.3 to 0.5 m [1 to 1.5 ft]
Radius = 0.6 to 1.0 m [2 to 3 ft]
0.6 to 1.0 m [2 to 3 ft] Offset
Shoulder
Shoulder Offset
Main Line Highway

Large

– C –

C.7.f.2 Divisional Islands

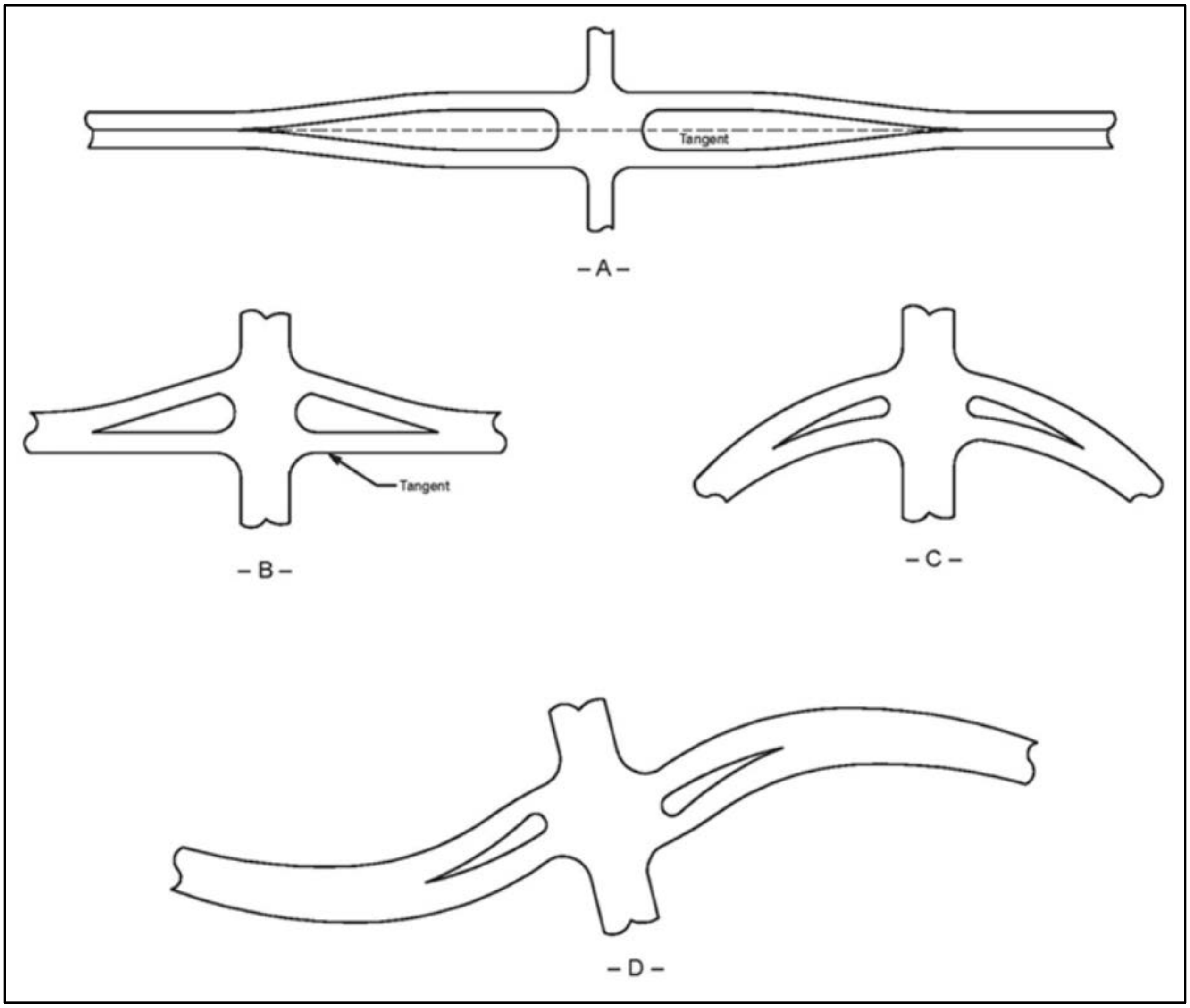
Divisional islands often are introduced on undivided highways at intersections. They alert drivers to the crossroad ahead and regulate traffic through the intersection. These islands are particularly advantageous in controlling left turns at skewed intersections and at locations where separate roadways 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 - xx~~27~~ 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 – xx~~27~~ may be used

Figure 3 – 10~~2~~ Alignment for Divisional Islands at Intersections



C.7.f.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~~3~~ 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~~s a~~ divisional islands that support~~s~~ a midblock crosswalk with stop and yield conditions~~ between transit stops~~. The distance A shown in the figures is based upon the ***MUTCD***.

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 C.10.a.4*** 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~~3~~Pedestrian Refuge Island

North Main Street, Gainesville, FL

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

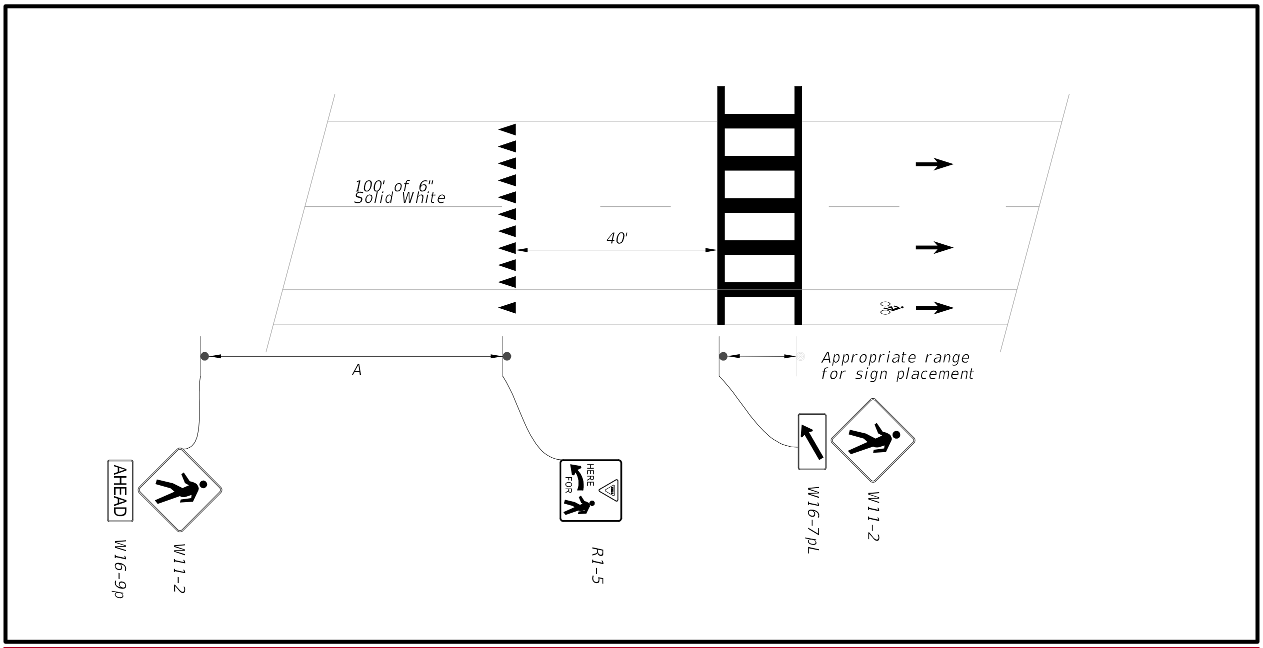
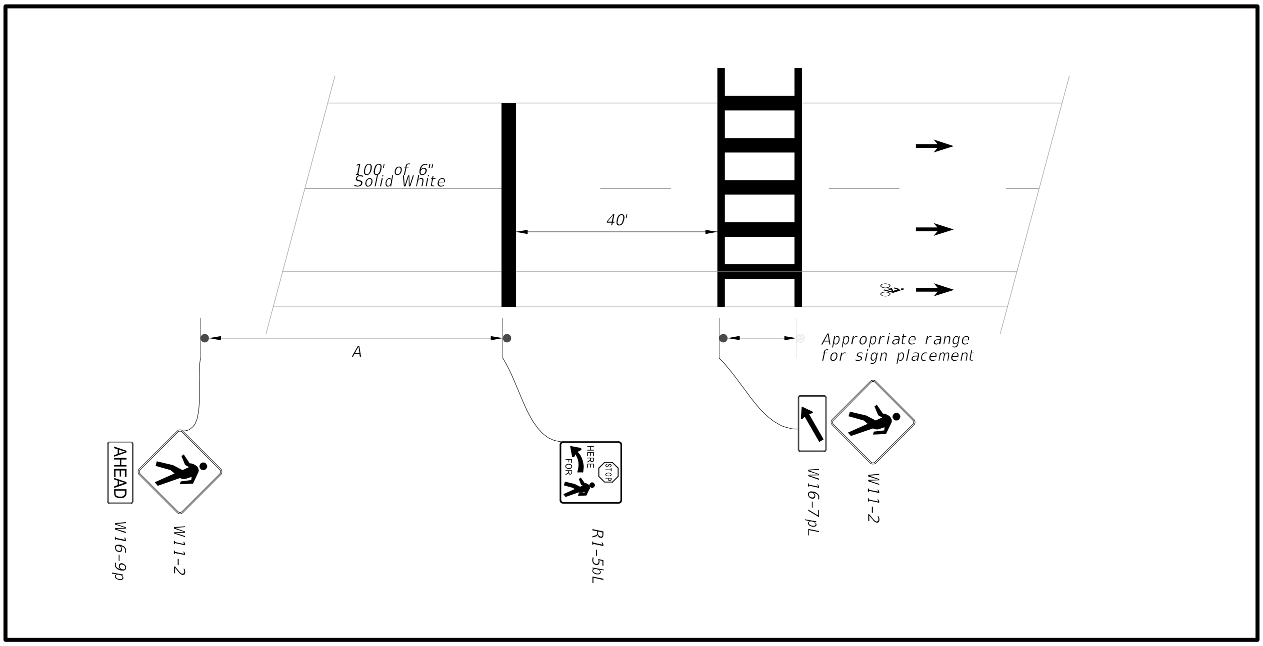


Figure 3 – 13 Pedestrian Crossing with Refuge Island (Stop Condition)



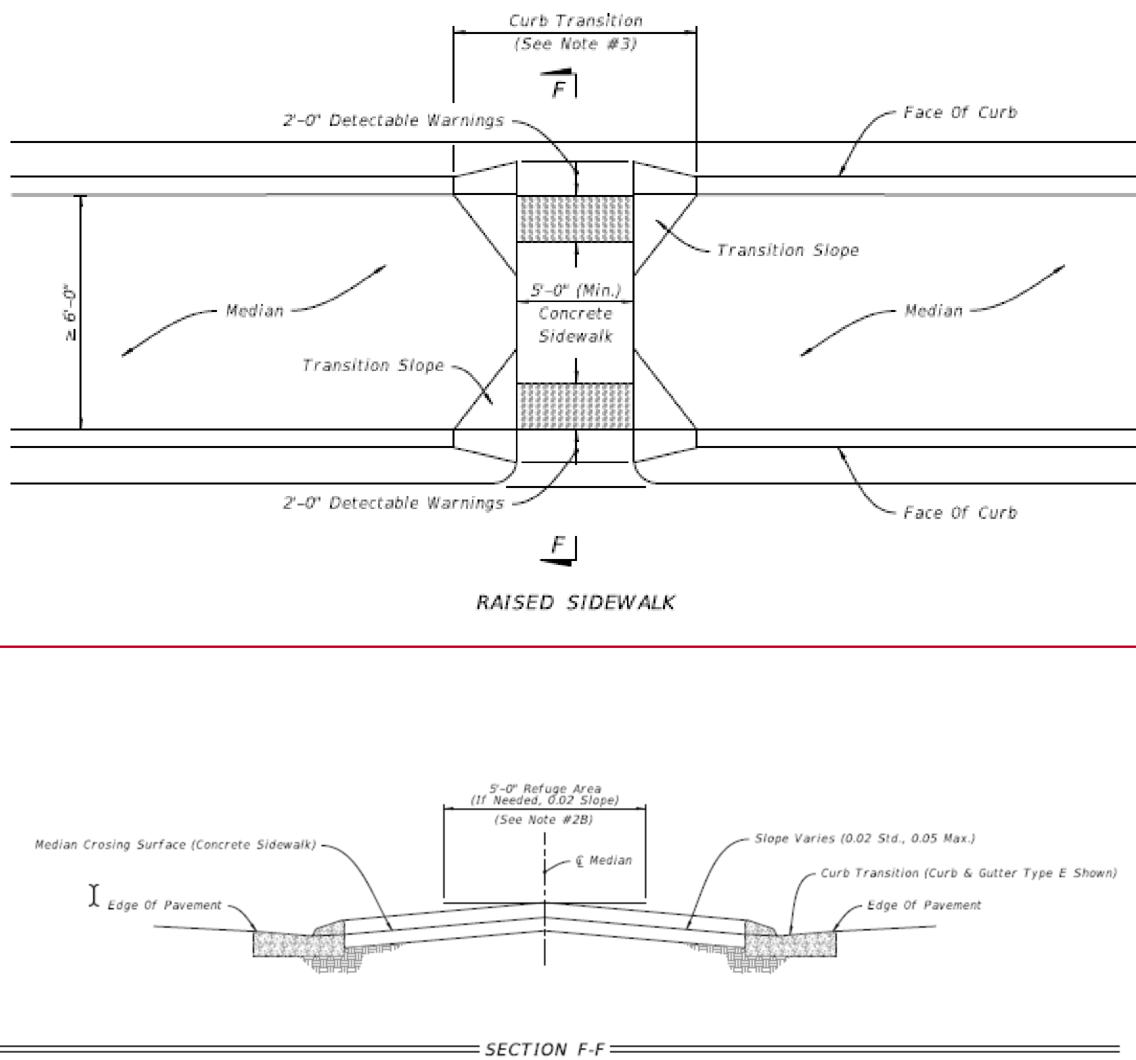
The distance A shown in Figures 3 – 12 and 3 – 13 for the advance warning sign should be:

Posted Speed	Advance Placement Distance (feet)
25 or Less	100
26 to 35	100
36 to 45	175

Source: 2009 MUTCD, with 2012 Revisions, Tale 2C-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 Department's ***Standard Plans 522-002 Detectable Warnings and Sidewalk Curb Ramps***.

Figure 3 – 14 Pedestrian Crossing in Refuge Island



~~C.7.gf Roadside Slopes, Clear Zone and Lateral Offset~~

~~The roadside clear zone is that area outside the traveled way available for use by errant vehicles. Vehicles frequently leave the traveled way during avoidance maneuvers, due to loss of control by the driver (e.g., falling asleep) or due to collisions with other vehicles. The primary function of the clear zone is to allow space and time for the driver to retain control of his vehicle and avoid or reduce the consequences of collision with roadside objects. This area also serves as an emergency refuge location for disabled vehicles.~~

~~The design of the roadway must also provide for adequate drainage of the roadway. Drainage swales within the clear zone should be gently rounded and free of discontinuities. Where large volumes of water must be carried, the approach should be to provide wide, rather than deep drainage channels. Side slopes and drainage swales that lie within the clear zone should be free of protruding drainage structures (***Chapter 4 - Roadside Design, D.6.c. Culverts***).~~

~~In the design of the roadside, the designer should consider the consequences of a vehicle leaving the traveled way at any location. It should always be the policy that protection of vehicles and occupants shall take priority over the protection of roadside objects. Further criteria and requirements for safe roadside design are given in ***Chapter 4 - Roadside Design***.~~

~~C.7.g.1 Clear Zone~~

~~Clear zone is the unobstructed, traversable area beyond the edge of the traveled way for the recovery of errant vehicles. The clear zone includes shoulders and bicycle lanes. The clear zone must be free of aboveground fixed objects, water bodies and non-traversable or critical slopes. Clear zone width requirements are dependent on AADT, design speed, and roadside slope conditions. With regard to the ability of an errant vehicle to traverse a roadside slope, slopes are classified as follows:~~

1. ~~Recoverable Slope – Traversable Slope 1:4 or flatter. Motorists who encroach on recoverable foreslopes generally can stop their vehicles or slow them enough to return to the roadway safely.~~
2. ~~Non-Recoverable Slope – Traversable Slope steeper than 1:4 and flatter than 1:3. Non-recoverable foreslopes are traversable but most vehicles will not be able to stop or return to the roadway easily. Vehicles on such slopes typically can be expected to reach the bottom.~~
3. ~~Critical Slope – Non-Traversable Slope steeper than 1:3. A critical foreslope is one on which an errant vehicle has a higher propensity to overturn.~~

~~Clear zone widths for recoverable foreslopes 1V:4H and flatter are provided in Table 3-15 Minimum Width of Clear Zone. Clear zone is applied as shown in Figures 3 - 14 Clear Zone Plan View and 3 – 15 Basic Clear Zone Concept.~~

~~On non-recoverable slopes steeper than 1:4 and flatter than 1:3, a high percentage of encroaching vehicles will reach the toe of these slopes. Therefore, the clear-zone distance cannot logically end at the toe of a non-recoverable slope. When such non-recoverable slopes are present within the clear zone width provided in Table 3-15, additional clear zone width is required. The minimum amount of additional width provided must equal the width of the non-recoverable slope with no less than 10 feet of recoverable slope provided at the toe of the non-recoverable slope. See Figure 3 – 16 Adjusted Clear Zone Concept.~~

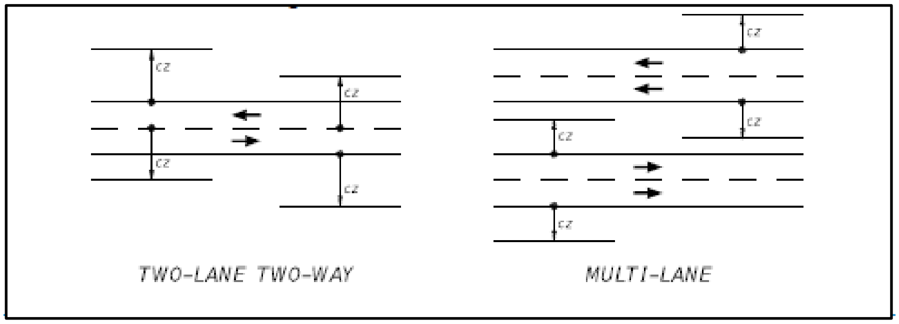
~~When clear zone requirements cannot be met, see Chapter 4 – Roadside Design for requirements for roadside barriers and other treatments for safe roadside design. In addition, the Department's ***Standard Plans, FDOT Design Manual***, ***AASHTO Roadside Design Guide (2011)***, and ***AASHTO Guidelines for Geometric Design of Very Low Volume Local Roads (ADT ≤ 400) (2001)*** may be referenced for a more thorough discussion of roadside design.~~

~~**Table 3 – 17 Minimum Width of Clear Zone**~~

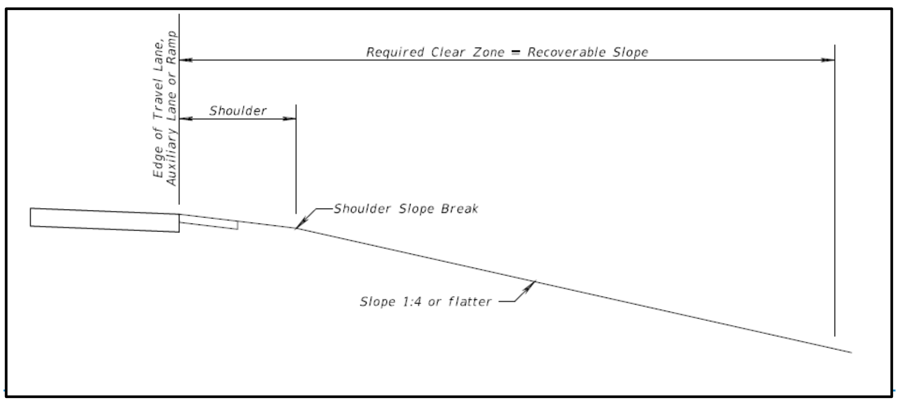
~~Clear Zone Widths – Feet~~						
~~Design Speed mph~~	~~AADT ≥ 1500~~			~~AADT < 1500~~[1]		
	~~Travel Lanes & Multilane Ramps~~		~~Aux Lanes and Single Lane Ramps~~	~~Travel Lanes & Multilane Ramps~~		~~Aux Lanes and Single Lane Ramps~~
	~~1V:6H or flatter~~	~~1V:5H to 1V:4H~~	~~1V:4H or flatter~~	~~1V:6H or flatter~~	~~1V:5H to 1V:4H~~	~~1V:4H or flatter~~
~~≤ 40~~	~~14~~	~~16~~	~~10~~	~~10~~[2]	~~12~~[2]	~~10~~[2]
~~45 – 50~~	~~20~~	~~24~~	~~14~~	~~14~~	~~16~~	~~14~~
~~55~~	~~22~~	~~26~~	~~18~~	~~16~~	~~20~~	~~14~~
~~60~~	~~30~~	~~30~~	~~24~~	~~20~~	~~26~~	~~18~~
~~65 – 70~~	~~30~~	~~30~~	~~24~~	~~24~~	~~28~~	~~18~~

1. ~~Clear Zone for roads functionally classified as Local Roads with a design AADT ≤ 400 vehicles per day:~~
    a. ~~A clear zone of 6 feet or more in width must be provided if it can be done so with minimum social/environmental impacts.~~
    b. ~~Where constraints of cost, terrain, right of way, or potential social/environmental impacts make the provision of a 6 feet clear zone impractical, clear zones less than 6 feet in width may be used, including designs with 0 feet clear zone.~~
    c. ~~In all cases, clear zone must be tailored to site-specific conditions, considering cost-effectiveness and safety tradeoffs. The use of adjustable clear zone widths, such as wider clear zone dimensions at sharp horizontal curves where there is a history of run-off-road crashes, or where there is evidence of vehicle encroachments such as scarring of trees or utility poles, may be appropriate. Lesser values of clear zone width may be appropriate on tangent sections of the same roadway.~~
    d. ~~Other factors for consideration in analyzing the need for providing clear zones include the crash history, the expectation for future traffic volume growth on the facility, and the presence of vehicles wider than 8.5 feet and vehicles with wide loads, such as farm equipment.~~
2. ~~May be reduced to 7 feet for a design AADT < 750 vehicles per day.~~

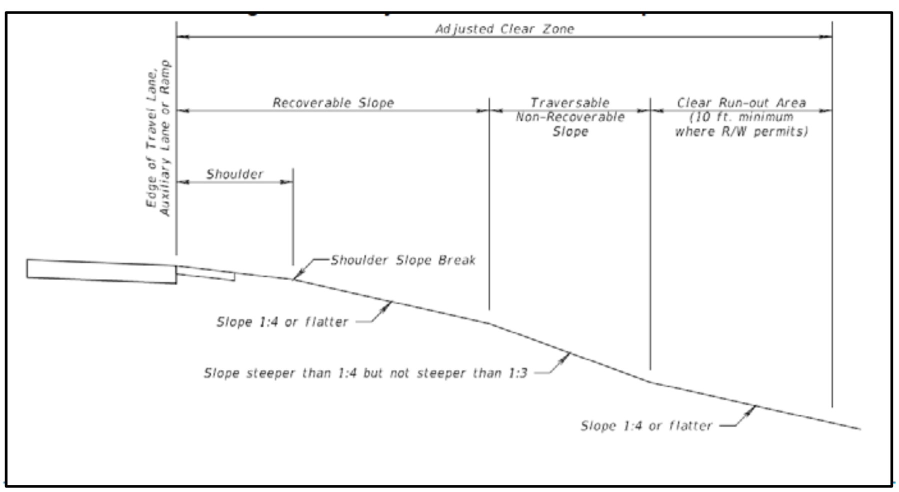
~~Figure 3 – 13 Clear Zone Plan View~~



~~Figure 3 – 14 Basic Clear Zone Concept~~

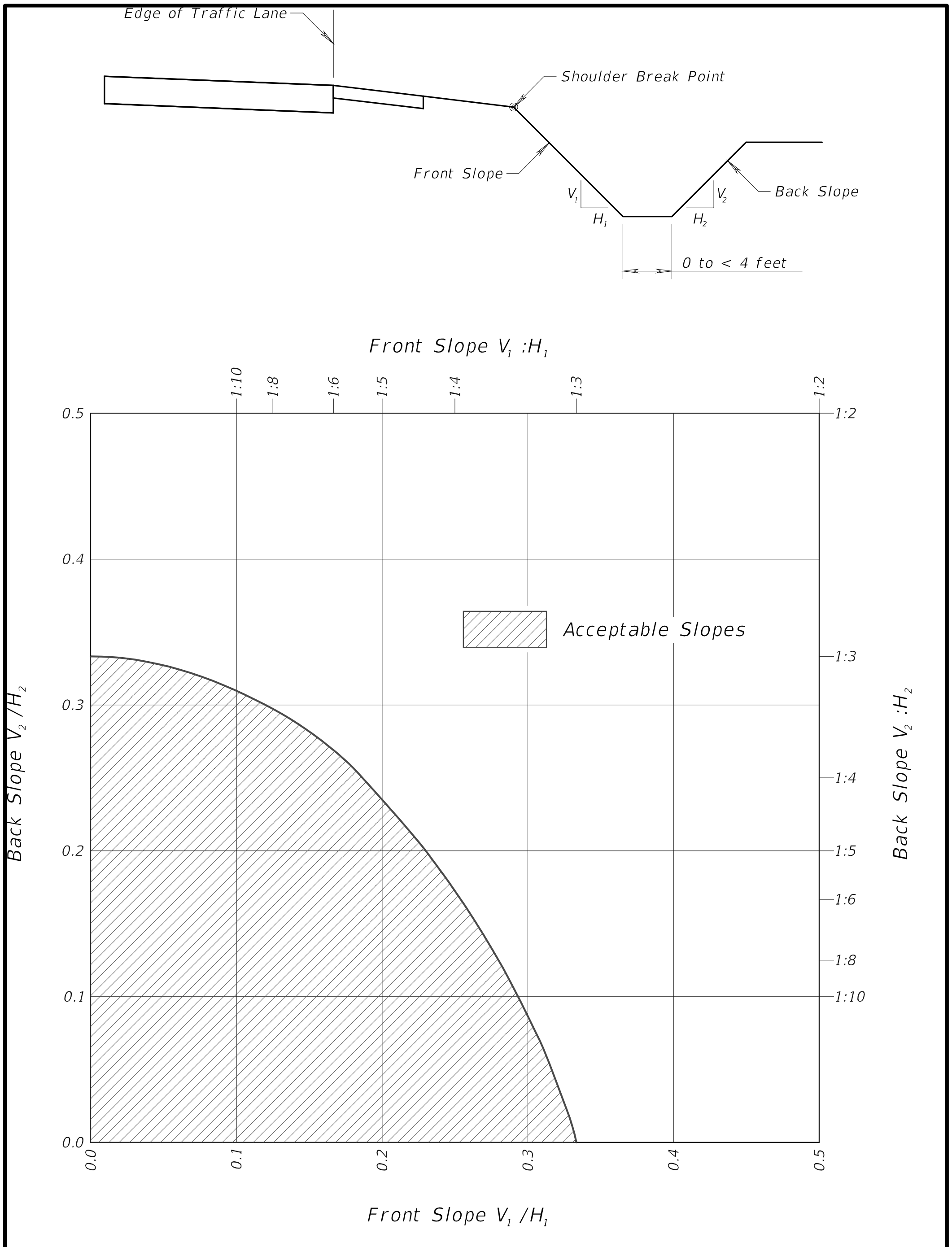


~~Figure 3 – 15 Adjusted Clear Zone Concept~~



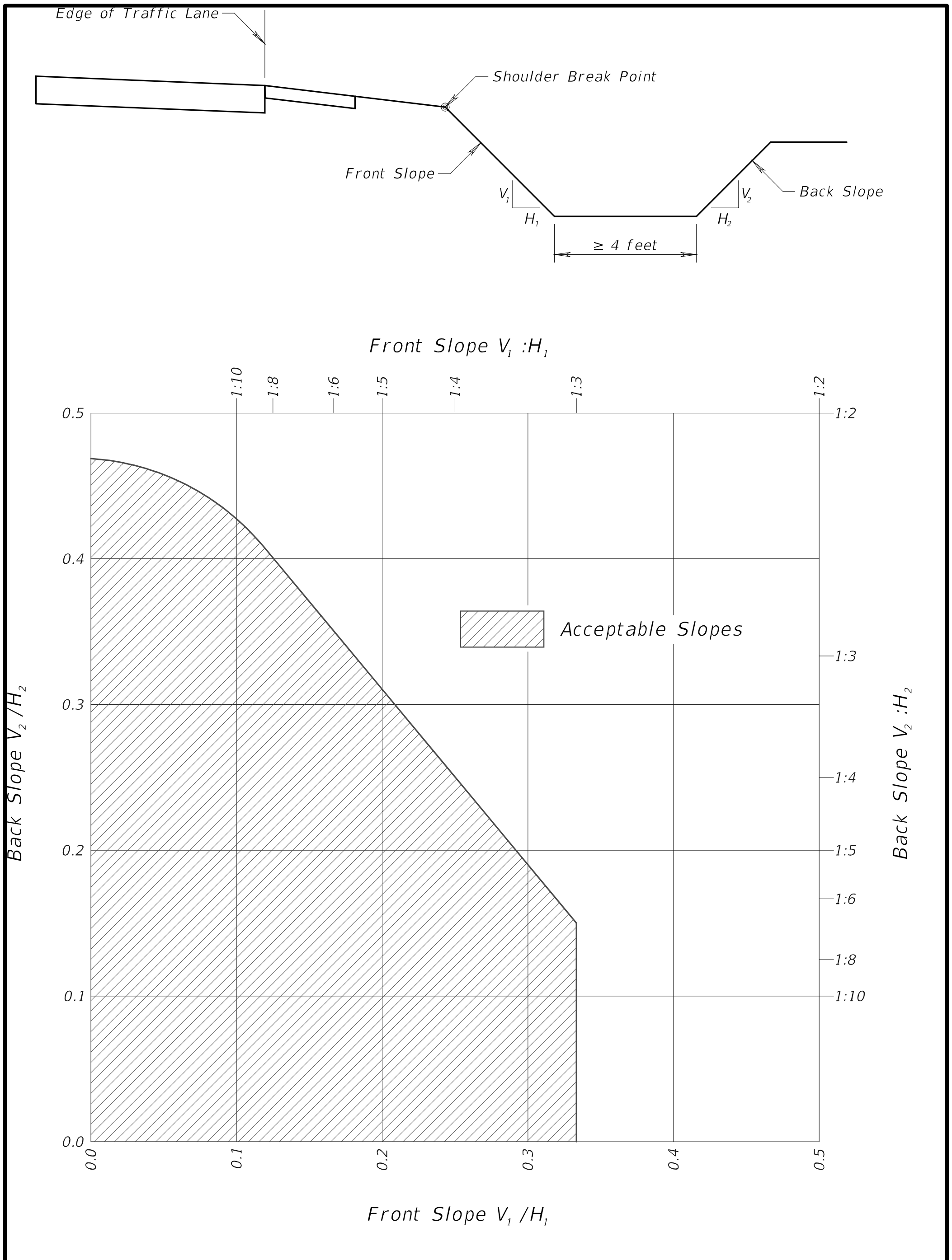
~~Roadside ditches may be included within the clear zone if properly designed to be traversable. Acceptable cross section slope criteria for roadside ditches within the clear zone is provided in Figure 3 - 17 Roadside Ditches – Bottom Width 0 to 4 Feet and Figure 3 - 18. These roadside ditch configurations are considered traversable.~~

~~Figure 3 – 16 Roadside Ditches – Bottom Width < 4 Feet~~



Ref: Figure 3-6, 2011 AASHTO Roadside Design Guide, 4th Edition

~~Figure 3 – 17 Roadside Ditches – Bottom Width ≥ 4 Feet~~



Ref: Figure 3-6, 2011 AASHTO Roadside Design Guide, 4th Edition

~~C.7.g.2 Lateral Offset~~

~~Lateral offset is the lateral distance from a specified point on the roadway such as the edge of traveled way or face of curb, to a roadside feature or above ground object that is more than 4 inches above grade. Lateral offset requirements apply to all roadways. The requirements for various objects or features are based on:~~

- ~~Design speed,~~
- ~~Location; i.e. rural areas or within urban boundary,~~
- ~~Flush shoulder or with curb,~~
- ~~Traffic volumes, and~~
- ~~Lane type; e.g. travel lanes, auxiliary lanes, and ramps.~~

~~Lateral Offset requirements are provided in Table 3-16.~~

~~Flush shoulder roadways typically have sufficient right of way to provide the required clear zone widths. Therefore, lateral offset requirements for these type roadway are based on providing the clear zone widths provided in Table 3-15.~~

~~On urban curbed roadways with design speeds ≤ 45 mph, lateral offsets based on Table 3-15 clear zone requirements should be provided where practical. However, these urban low speed roads are typically located in areas where right of way is restricted (characterized by more dense abutting development, presence of parking, closer spaced intersections and accesses to property, and more bicyclists and pedestrians). The available right of way is typically insufficient to provide the required clear zone widths. Therefore, lateral offset requirements for above ground objects on these roadways are based on offsets needed for normal operation and not on maintaining a clear roadside for errant vehicles.~~

~~Table 3 – 18 Lateral Offset~~

~~Lateral Offset (feet)~~		
~~Roadside Feature~~	~~Urban Curbed Roadways Design Speed ≤ 45 mph~~	~~All Other~~
~~Above Ground Objects[1]~~	~~4 ft from Face of Curb[2]~~	~~Clear Zone Width~~
~~Drop Off Hazards[3]~~	~~Clear Zone Width~~	~~Clear Zone Width~~
~~Water Bodies and Canal Hazards~~	~~See Chapter 4~~	~~See Chapter 4~~

1. ~~Aboveground objects are anything greater than 4 inches in height and are firm and unyielding or do not meet crashworthy or breakaway criteria. For urban curbed areas ≤ 45 mph this also includes crashworthy or breakaway objects except those necessary for the safe operation of the roadway.~~
2. ~~May be reduced to 1.5 ft. from Face of Curb on roads functionally classified as Local Streets and on all roads where the 4 ft. minimum offset cannot be reasonably obtained and other alternatives are deemed impractical.~~
3. ~~Drop off hazards are:~~
   a. ~~Any vertical faced structure with a drop off (e.g. retaining wall, wing-wall, etc.) located within the Clear Zone.~~
   b. ~~Slopes steeper than 1:3 located within the Clear Zone.~~
   c. ~~Drop-offs with significant crash history.~~

~~The clear zone width is defined as follows:~~

- Flush Shoulder~~Rural~~ Ssections - measured from the edge of the outside motor vehicular traveled way
- Urban Curbed Ssections ≤ 45 mph - measured from the face of the curb

~~The minimum permitted widths are provided in Table 3 - 13. These are minimum values only and should be increased wherever practical.~~

~~In rural areas, it is desirable, and frequently economically feasible, to increase the width of the clear zone. Where traffic volumes and speeds are high, the width should be increased. The clear zone on the outside of horizontal curves should be increased due to the possibility of vehicles leaving the roadway at a steeper angle.~~

~~Table 3 – 15~~
~~Minimum Width of Clear Zone~~

~~Type of Facility~~	~~DESIGN SPEED (mph)~~							
	~~25 and Below~~	~~30~~	~~35~~	~~40~~	~~45~~	~~50~~	~~55~~	~~60 and Above~~
	~~MINIMUM CLEAR ZONE (FEET)~~							
~~Flush Shoulder~~•	~~6~~	~~6 Local~~ ~~10 Collectors~~ ~~14 Arterials~~	~~6 Local~~ ~~10 Collectors~~ ~~14 Arterials~~	~~10 Collectors~~ ~~14 Arterials~~	~~14 Arterials and Collectors ADT < 1500~~ ~~18 Arterials and Collectors ADT ≥ 1500~~	~~14 Arterials and Collectors ADT < 1500~~ ~~18 Arterials and Collectors ADT ≥ 1500~~	~~18 Arterials and Collectors ADT < 1500~~ ~~24 Arterials and Collectors ADT ≥ 1500~~	~~18 Arterials and Collectors ADT < 1500~~ ~~30 Arterials and Collectors ADT ≥ 1500~~
~~Curbed~~ *	~~1 ½~~	~~4~~ **	~~4~~ **	~~4~~ **	~~4~~ **	~~N/A~~ ••	~~N/A~~ ••	~~N/A~~ ••

~~* From face of curb.~~

~~** On projects where the 4 foot minimum offset cannot be reasonably obtained and other alternatives are deemed impractical, the minimum may be reduced to 1 ½'.~~

~~• Use rural for urban facilities when no curb and gutter is present. Measured from the edge of through travel lane on rural section.~~

~~•• Curb and gutter not to be used on facilities with design speed > 45mph.~~

~~NOTE: ADT in Table 3-13 refers to Design Year ADT.~~

~~C.7.g.3 Roadside Slopes~~

~~The slopes of all roadsides should be as flat as possible to allow for safe traversal by out of control vehicles. A slope of 1:4 or flatter should be used, desirably 1:6 or flatter. The transition between the shoulder and adjacent side slope should be rounded and free from discontinuities. A slope as steep as 1:3 may be used within the clear zone if the clear zone width is adjusted to provide a clear runout area as described in C.7.f.2. The side slopes should be reduced flatter on the outside of horizontal curves.~~

~~Where roadside ditches or cuts require backslope, slopes shall conform to acceptable slope conditions shown in Figures 3-17 and 3-18. The desirable backslope is 1:4 or flatter. Ditch bottoms should be at least 4 feet wide and can be flat or gently rounded.~~

~~C.7.gf.43 Criteria for Guardrail~~

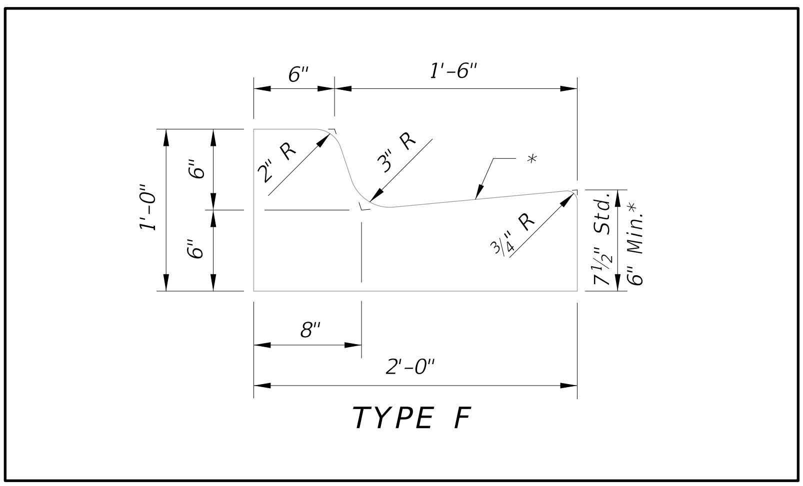
~~If space and economic constraints are severe, it is permissible, but not desirable, to use guardrails in lieu of the requirements for width and slope of clear zone. Where the previously described requirements for clear zone are not met, guardrails (or other longitudinal barriers) should be considered. Guardrails should also be considered for protection of pedestrian pathways or protection from immovable roadside hazards.~~

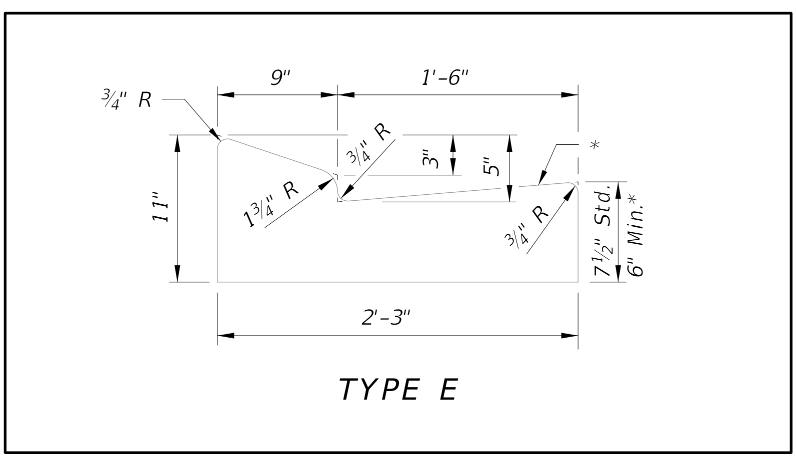
~~The general policy to be followed is that guardrails should be used if impact with the guardrail is less likely or considered less severe than impact with roadside objects. Further requirements and design criteria for guardrails are given in ***Chapter 4 - Roadside Design***.~~

C.7.g~~hg~~ 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. ~~Sloping curbs are used along the outside edge of the roadway to discourage vehicles from leaving the roadway~~. In Florida, the standard curb of this type is 6 inches in height. See Figure 3 – 15~~9~~ 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~~9~~ Standard Detail for FDOT Type F and E Curbs





C.7.h~~ih~~ Parking

~~When on-street parking is to be an element of design, parallel parking should be considered. Under certain circumstances, angle parking is an allowable form of street parking.~~ The type of on-street parking selected should depend on the specific function and width of the street, the adjacent land use, traffic volume, as well as existing and anticipated traffic operations.

On-street parking provides necessary parking supply, helps manage traffic speeds, and provides separation between the sidewalk and the travel lanes. 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. Parking may be either parallel or angle (traditional or reverse).

Parallel parking spaces are 8 feet wide, measured from the edge of the travel lane to the face of curb, and 22 feet long. Existing parallel parking lanes with a minimum width of 7 feet may remain. Angle parking spaces are 17 feet wide measured from the edge of the driving lane to the face of curb, with 45-degree angle stalls, 9 feet wide.

~~It can generally be stated that on-street parking decreases through capacity, impedes traffic flow, and increases crash potential. However, where parking is needed, and adequate off-street parking facilities are not available or feasible, on-street parking may be necessary.~~

C.7.i~~ji~~ 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 construction of future grade separations or other intersection improvements at selected crossroads.
- 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.

C.7.j~~kj~~ Changes in Typical Section

C.7.j~~kj~~.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.

C.7.~~.~~j~~kj~~.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 C.9.c.1 for additional information.

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

C.7.j~~kj~~.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*** provides further information on preferential lane markings. See ***Chapter 9 – Bicycle Facilities*** for information on marking bicycle lanes.

C.7.j~~kj~~.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 – 18 Minimum Shoulder Widths for Flush Shoulder

Rural Highways~~Table 3 – 11~~. 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.

C.7.j~~kj~~.4.(a) Lateral Offset~~Horizontal Clearance~~

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.

C.7.j~~kj~~.4.(b) 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.

C.7.j~~kj~~.4.(c) 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***.

C.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 ***Department's Driveway Information Guide (2008)*** and ***Median Handbook (2014)*** provide further information on designing roadways and connections to support access management.

C.8.a Justification

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

C.8.b General Criteria

C.8.b.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~~near~~ the influence zone of intersections or other points that would tend to produce traffic conflict.

C.8.b.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.

C.8.b.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.

C.8.b.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.

C.8.b.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.

C.8.b.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***, 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 are 20 - 25 mph or less, consider urban, suburban, rural, single vs, multilane.

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

C.8.c 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 – 21~~1716~~ 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 – 23~~17~~6 Access Control for All Limited Access Highways

	Urban	Rural
Minimum Spacing		
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	

C.8.d 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.

C.8.e 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 2 - Land Development***.

C.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~~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.

C.9.a 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 C.8 Access Control, this chapter.
- The requirements for bicycle and pedestrian movements should receive special consideration.

C.9.b 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.

C.9.b.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 C.9.B.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.

C.9.b.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 – 17~~Figure 3 - 11~~ Departure Sight Triangle in Section C.9.b.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.

C.9.b.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.

C.9.b.3.(a) 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 – 22 Minimum Stopping Sight Distance (Rounded Values)~~Table 3 - 1~~8~~7~~ Sight Distance for Approach to Stops. 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 - 10~~Figure 3 - 16~~0~~ 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.

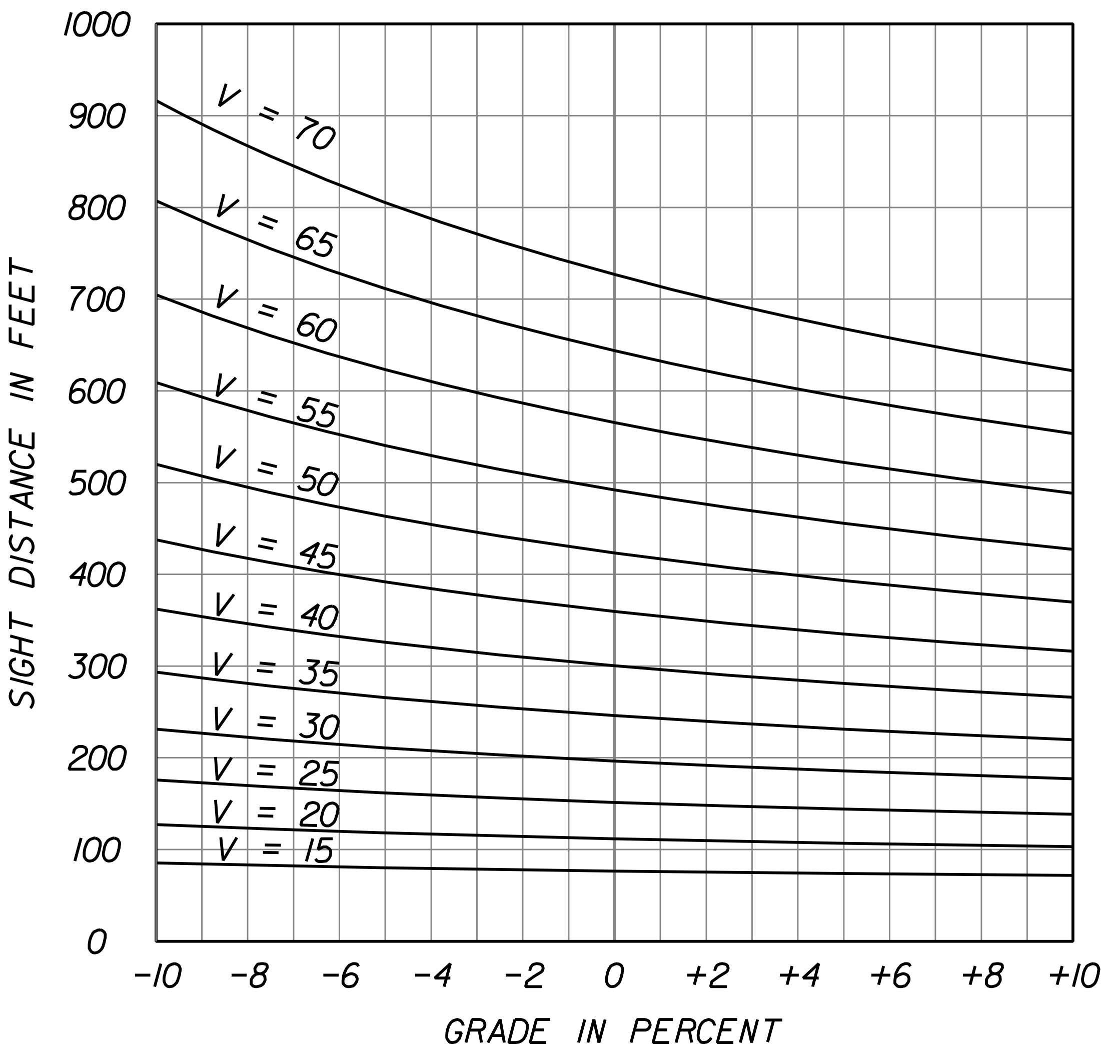
Table 3 – 24~~1~~8~~7~~ Minimum Stopping Sight Distance ~~Sight Distance for Approach to Stops~~
(Rounded Values)

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

C.9.b.3.(b) 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 – 22 Minimum Stopping Sight Distance (Rounded Values)~~Table 3 - 18~~8~~7~~ or Figure 3 – 16 Sight Distances for Approach to Stop on Grades~~Figure 3 -~~ 2~~10~~. 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 – 16~~1~~200
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

C.9.b.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 - 17~~2~~~~11~~ Departure Sight Triangle (Traffic Approaching from Left or Right) and 3 - 18~~2~~~~12~~ 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):

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

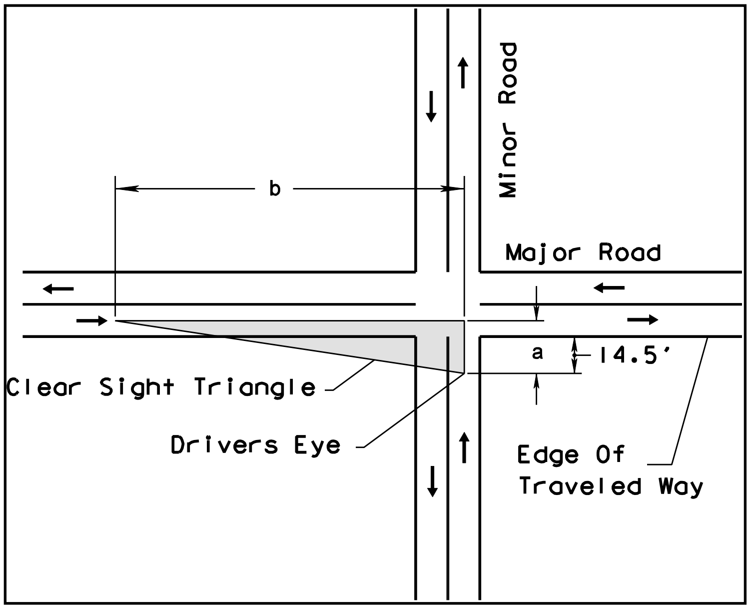
See Sections C.9.b.4.(c) and (d) for design time gaps for Case B.

For Intersections with Traffic Signal Control see Section C.9.b.4.(e) (AASHTO Case D).

For intersections with all way stop control see Section C.9.b.4.(f) (AASHTO Case E).

For left turns from the major road see Section C.9.b.4.(g) (AASHTO Case F).

Figure 3 – 17~~211~~
Departure Sight Triangle
(Traffic Approaching from Left or Right)



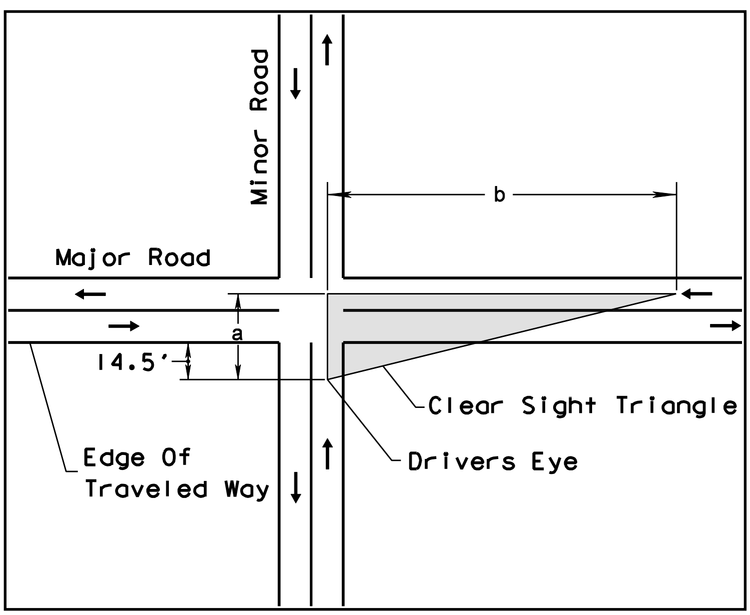
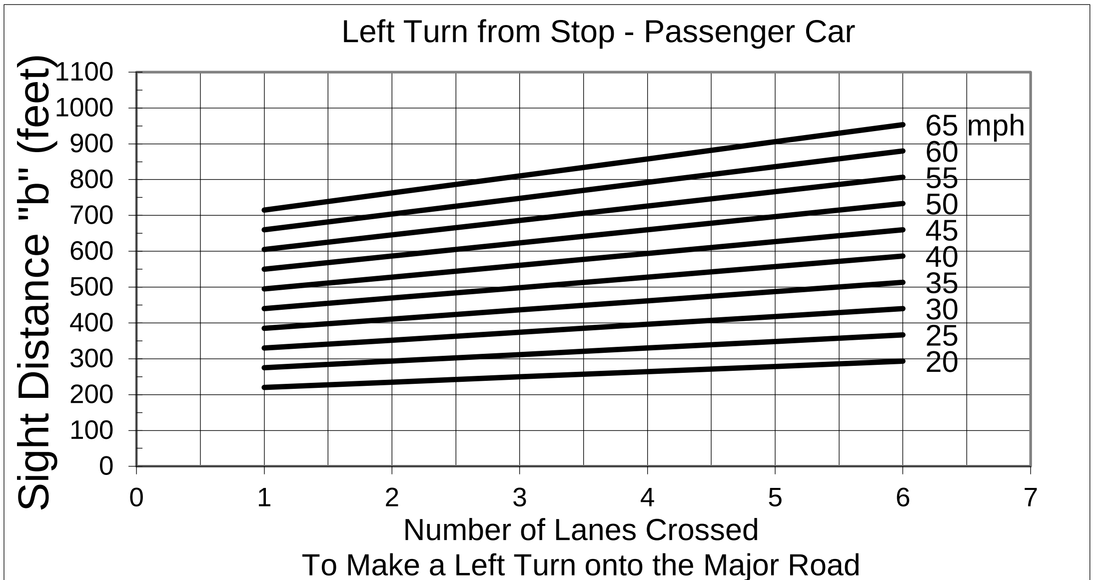


Figure 3 – 18~~2~~12
Intersection Sight Distance



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.

C.9.b.4.(a) 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.

C.9.b.4.(b) 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.

C.9.b.4.(c)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.

C.9.b.4.(d) 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.

C.9.b.4.(e)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.

C.9.b.4.(f) 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.

C.9.b.4.(g) 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 - 19~~213~~ Sight Distance for Vehicle Turning Left from Major Road).

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

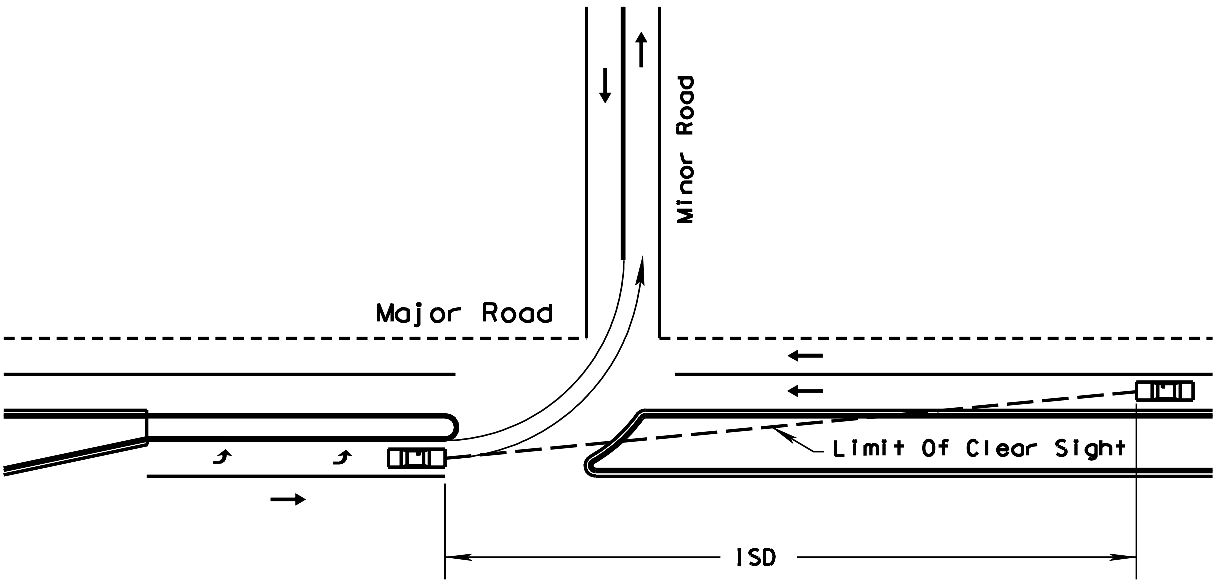
Design Vehicle	Time Gap (t_g) in Seconds
Passenger Car	5.5
Single Unit Truck	6.5
Combination Truck	7.5

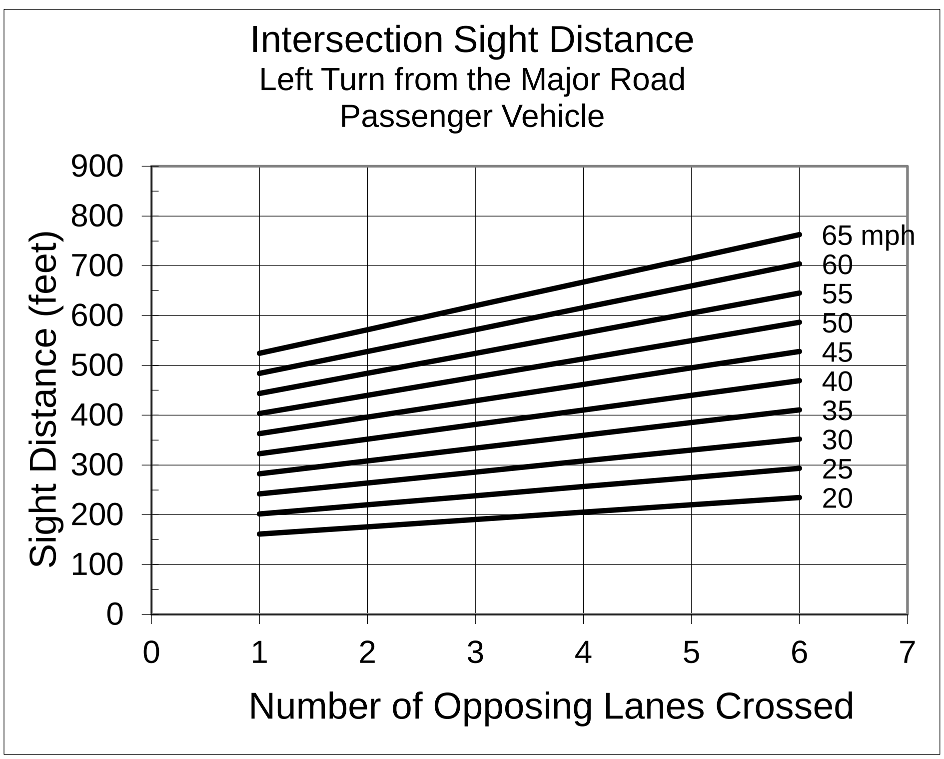
For left turning vehicles that cross more than one opposing lane, add 0.5 seconds for passenger cars and 0.7 seconds for trucks for each additional lane to be crossed.

C.9.b.4.(h) Intersection Sight Distance References

The Department's ***Design Manual, Chapter 212 Intersections*** ~~Standards, Index 546~~, 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.

Figure 3 – 19~~213~~
Sight Distance for Vehicle Turning Left from Major Road





C.9.c 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 C.8 Access Control, this chapter. The pavement width and cross slopes of auxiliary lanes should meet the minimum requirements shown in Table 3~~8~~-17 Minimum Lane Widths.

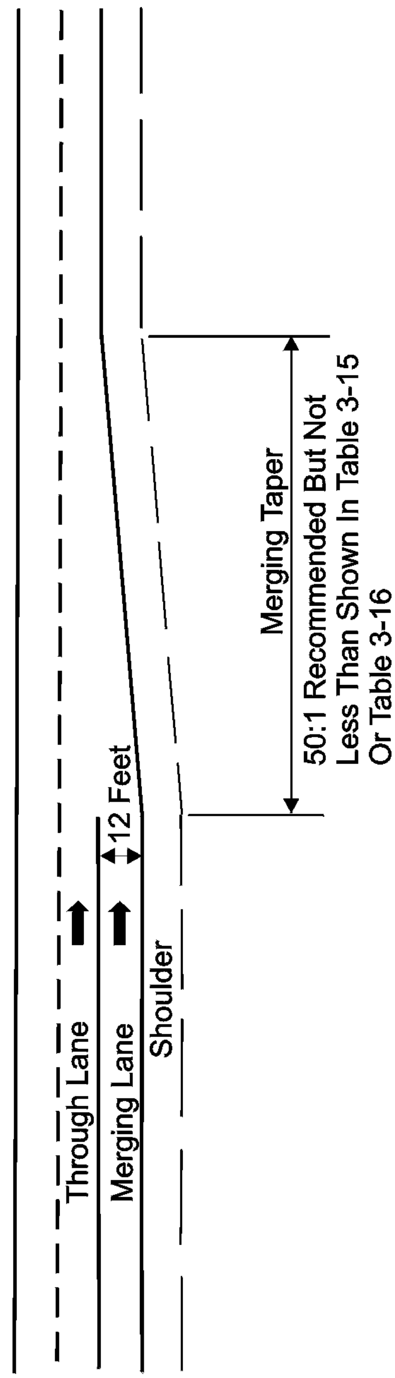
C.9.c.1 Merging Maneuvers

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 - 23~~198~~ 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 - 20~~14~~ 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 – 25~~198~~ 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 – 20~~14~~
Termination of Merging Lanes



C.9.c.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 - 24~~019~~ Design Lengths of Speed Change Lanes Flat Grades. Where acceleration occurs on a grade, the required distance is obtained by using Tables 3 - 25~~019~~ and 3 - 26~~10~~ Ratio of Length of Speed Change Lane on Grade to Length on Level.

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 - 20~~14~~ Termination of Merging Lanes), not less than that length set forth in Table 3 - 23~~198~~ 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 - 20~~14~~. Recommended acceleration lanes for freeway entrance terminals are given in Table 3 - 26~~21~~ Minimum Acceleration Lengths for Entrance Terminals.

Table 3 – 26~~019~~ Design Lengths of Speed Change Lanes Flat Grades - 2 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

* For urban street auxiliary lanes, shorter tapers may be used due to lower operating speeds. Refer to Figure 3-16 for allowable taper rates.

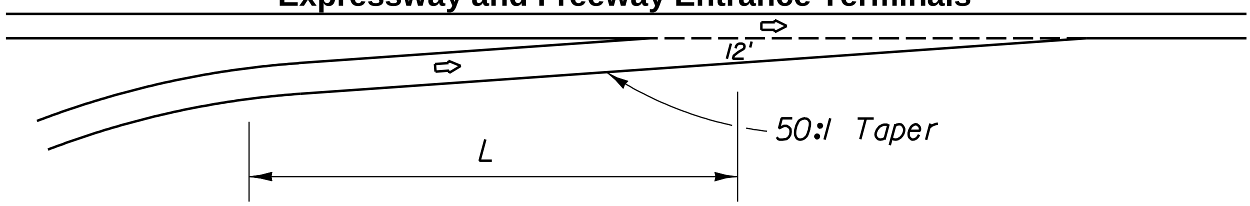
Table 3 – 27~~10~~ Ratio of Length of Speed Change Lane on Grade to Length on Level

Deceleration Lane			Acceleration Lane					
	Design Speed of Turning Roadway (mph)			Design Speed of Turning Roadway (mph)				
Design Speed of Highway (mph)	All Speeds 3% -4% Upgrade	All Speeds 3%-4% Downgrade	Design Speed of Highway (mph)	20	30	40	50	All Speeds
				3% - 4% Upgrade				3% - 4% Downgrade
All Speeds	0.9	1.2	40	1.3	1.3	---	---	0.7
			45	1.3	1.35	---	---	0.675
			50	1.3	1.4	1.4	---	0.65
			55	1.35	1.45	1.45	---	0.625
			60	1.4	1.5	1.5	1.6	0.6
			65	1.45	1.55	1.6	1.7	0.6
			70	1.5	1.6	1.7	1.8	0.6
	5% - 6% Upgrade	5% - 6% Downgrade		5% - 6% Upgrade				5% - 6% Downgrade
All Speeds	0.8	1.35	40	1.5	1.5	---	---	0.6
			45	1.5	1.6	---	---	0.575
			50	1.5	1.7	1.9	---	0.55
			55	1.6	1.8	2.05	---	0.525
			60	1.7	1.9	2.2	2.5	0.5
			65	1.85	2.05	2.4	2.75	0.5
			70	2.0	2.2	2.6	3.0	0.5
Ratios in this table multiplied by the values in Table 3 – 18 give the length of speed change lane for the respective grade.								

Table 3 – 28~~21~~ Minimum Acceleration Lengths for Entrance Terminals

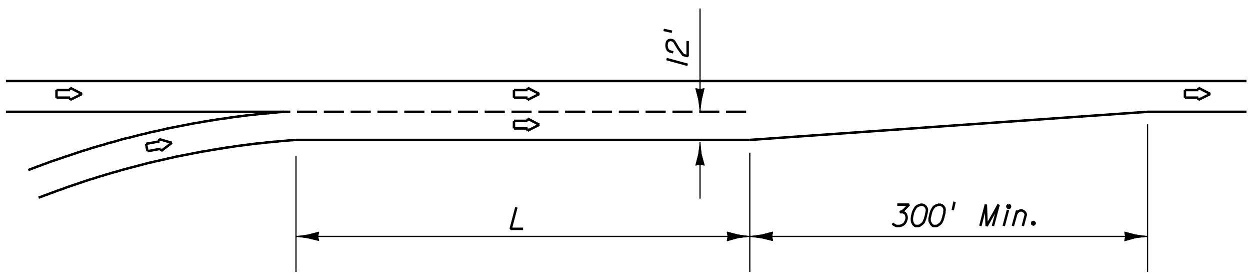
Highway Design Speed (mph)	**L = Acceleration Length (feet)**								
	For Entrance Curve Design Speed (mph)								
	Stop Condition	15	20	25	30	35	40	45	50
30	180	140	---	---	---	---	---	---	---
35	280	220	160	---	---	---	---	---	---
40	360	300	270	210	120	---	---	---	---
45	560	490	440	380	280	160	---	---	---
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.

C.9.c.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 - 27~~019~~. Minimum Deceleration Lengths for Exit Terminals.

The required distance for deceleration on grades is given in Tables 3 - 24~~019~~ and 3 - 25~~10~~.

The length of deceleration lanes shall be no less than the values obtained from Tables 3 - 24~~019~~ and 3 - 25~~10~~, 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 - 21~~15~~ Entrance for Deceleration Lane. The initial length of straight taper, shown in Table 3 - 26~~019~~, 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 - 27~~32~~ Minimum Deceleration Lengths for Exit Terminals.

- ~~Storage Lanes - Where exit lanes are required (C.8 Access Control, this chapter), or desirable on low speed streets and~~

~~highways, storage lanes may be used in place of or in conjunction with deceleration lanes. Storage lanes should be considered on all facilities. Although the primary function of storage lanes is to provide protection and storage for turning vehicles, it is desirable to provide sufficient length to allow for deceleration capabilities. Storage lanes should conform to the general configuration shown in Figure 3 - 216 Typical Storage Lane.~~

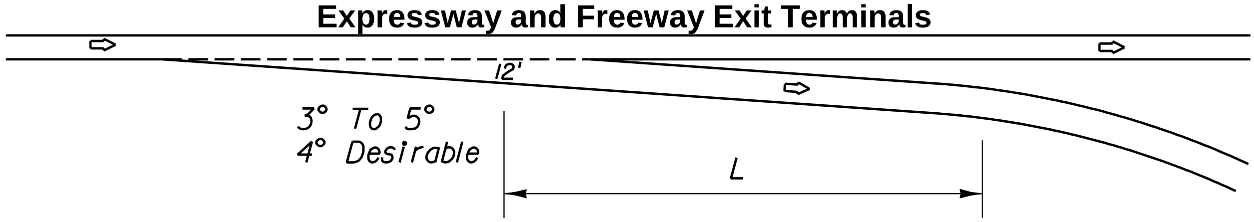
- ~~The length of storage lanes for unsignalized intersections may be obtained from the table in Figure 3 - 216. The full width portion of storage lanes should, where possible, be increased to allow for expected storage of vehicles (Table 3 - 2 for vehicle lengths). As a minimum requirement, storage for at least two passenger cars (40 - 50 feet) should be provided.~~

- ~~On collector or arterial streets (design speed 45 mph or less), tapers preceding storage lanes and approaching intersections at grade may be shorter than those given in Table 3 - 2019 (AASHTO for recommended lengths).~~

Table 3 – 29~~32~~ Minimum Deceleration Lengths for Exit Terminals

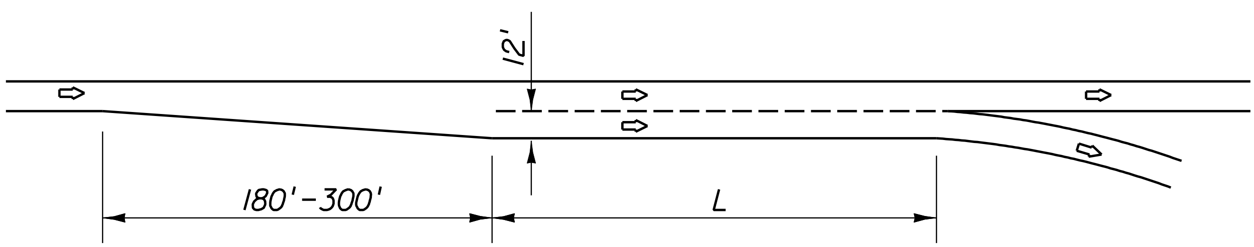
Highway Design Speed (mph)	L = Deceleration Length (feet)								
	For Design Speed of Exit Curve (mph)								
	Stop Condition	15	20	25	30	35	40	45	50
30	235	200	170	140	---	---	---	---	---
35	280	250	210	185	150	---	---	---	---
40	320	295	265	235	185	155	---	---	---
45	385	350	325	295	250	220	---	---	---
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

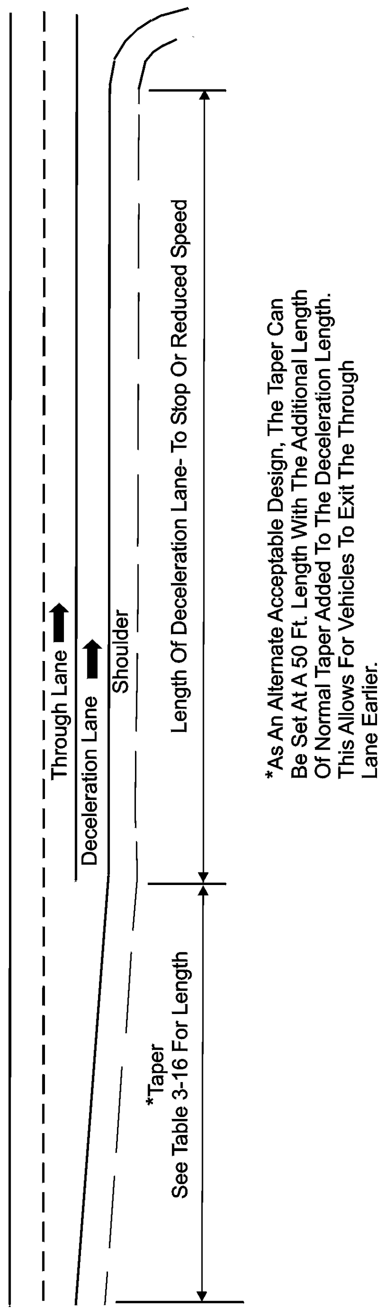
Recommended when design speed at exit curve is 50 mph or greater and when approach visibility is good.



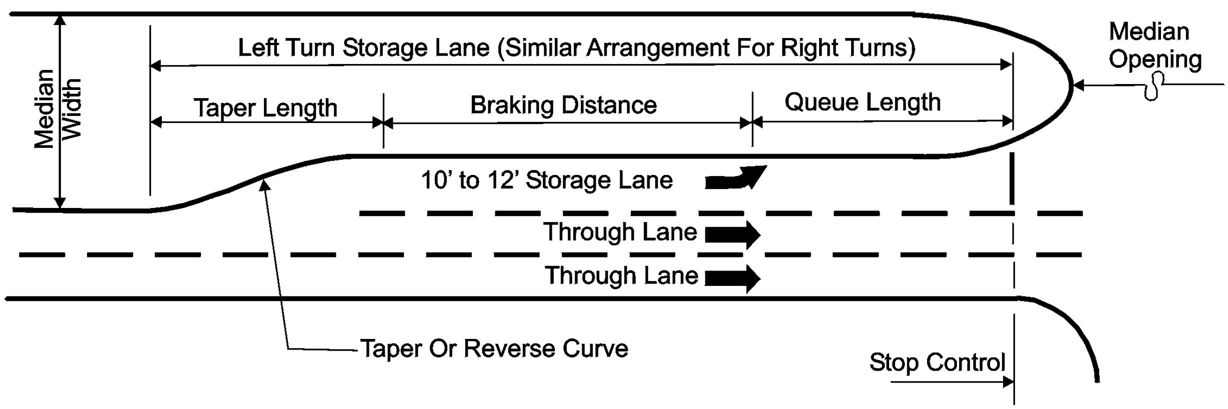
PARALLEL TYPE

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

Figure 3 – 21~~15~~
Entrance for Deceleration Lane



~~Figure 3 – 216~~
~~Typical Storage Lane~~



~~Storage Queue Length - Unsignalized Intersections~~

~~Turning Vehicles Per Hour~~	~~30~~	~~60~~	~~100~~	~~200~~	~~300~~
~~Required Storage Length (feet)~~	~~25~~	~~50~~	~~100~~	~~175~~	~~250~~

~~At signalized intersections, the required queue length depends on the signal cycle length, the signal phasing arrangement, and rate of arrivals and departures of turning vehicles.~~

~~In absence of a turning movement study, it is recommended that 100 ft. of queue length be provided in urban/suburban areas and 50 ft. of queue length be provided in rural/town areas as a minimum.~~

~~Taper Length And Braking Distance (feet)~~

~~Highway Design Speed (mph)~~	~~Storage Entry Speed* (mph)~~	~~Taper Length~~	~~Brake To Stop~~	
			~~Urban**~~	~~Rural***~~
~~35~~	~~25~~	~~70~~	~~75~~	~~---~~
~~40~~	~~30~~	~~80~~	~~75~~	~~---~~
~~45~~	~~35~~	~~85~~	~~100~~	~~---~~
~~50~~	~~40/44~~	~~105~~	~~135~~	~~215~~
~~55~~	~~48~~	~~125~~	~~---~~	~~260~~
~~60~~	~~52~~	~~145~~	~~---~~	~~310~~
~~65~~	~~55~~	~~170~~	~~---~~	~~350~~

~~* Reaction Precedes Entry~~
~~** Minimum Braking Distance, Wet Conditions~~
~~*** Customary Braking Distance, Wet Conditions~~

~~The storage lane may be in place of or in addition to deceleration length (See Section C.9.c.3).~~

C.9.c.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.

C.9.c.4.(a) Widths of Auxiliary Lanes

The minimum widths for auxiliary lanes are given in Table 3-17~~0~~ Minimum Lane Widths.

C.9.c.4.(b) Lengths of Auxiliary Lanes for Deceleration

Recommended lengths for auxiliary lanes for deceleration (turn lanes) at intersections are provided in Figure 3-26. These lengths are based on the Department's criteria. As shown in Figure 3-22~~6~~, 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 3-22~~6~~ 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 Department'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 - 22 Auxiliary Lanes for Deceleration at Intersections (Turn Lanes) and Table 3 –28 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).

Figure 3 – 22~~6~~ Auxiliary Lanes for Deceleration at Intersections (Turn Lanes)

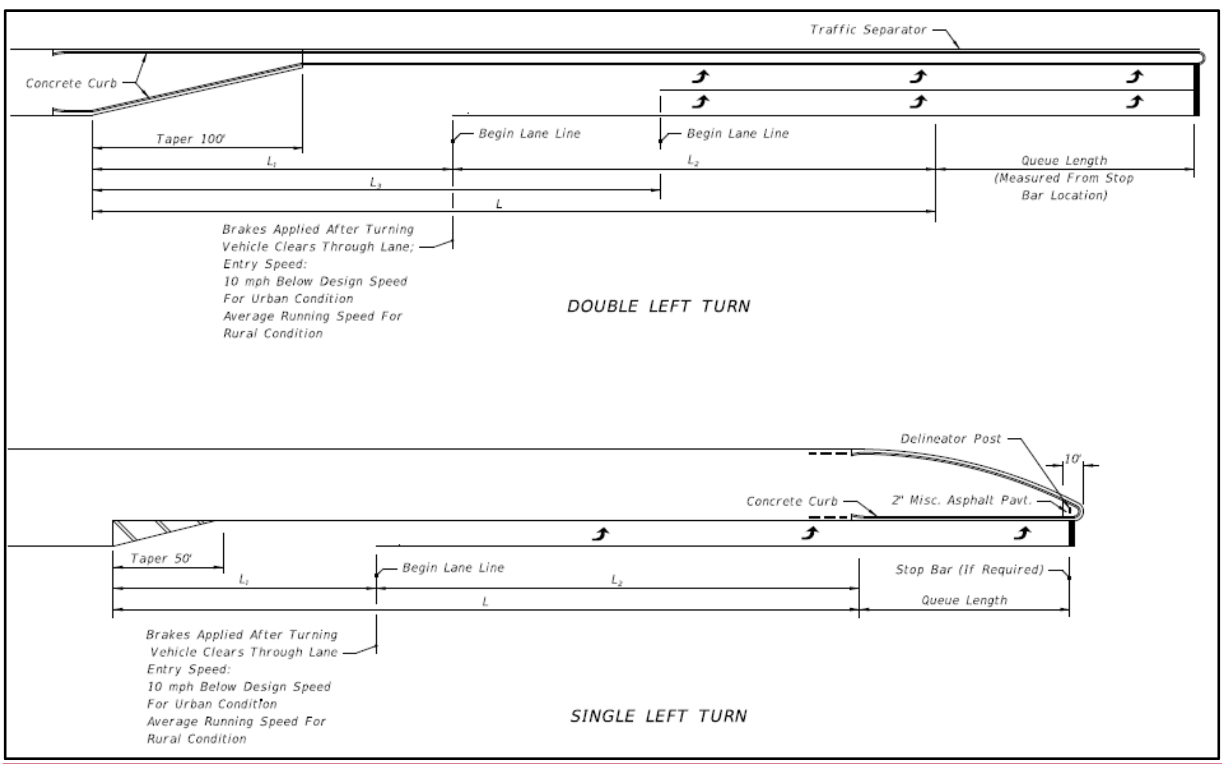


Table 3 – 30~~2~~ Turn Lanes – Curbed and Uncurbed Medians

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	60	75	135	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
Notes: 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.								

C.9.c.3.(c) 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 C.9.c.2 Acceleration Lanes.

C.9.d 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.

C.9.d.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.

C.9.d.2 Horizontal Alignment

- Curvature - The minimum permitted radii (maximum degree) of curvature for various values of superelevation are given in Table 3 - 29~~43~~ 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***, should also be considered.

Table 3 – 31~~2~~4~~3~~ 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.

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

- Superelevation Transition - Minimum superelevation transition (runoff) rates (maximum relative gradients) are given in Tables 3 - 30~~24~~ Maximum Rate of Change in Pavement Edge Elevation for Curves at Intersections and 3 - 31~~265~~ 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***, should also be considered.

Table 3 – 32~~254~~ 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 – 33~~265~~ 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

C.9.d.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 - 3~~187~~. For additional guidance on vertical alignment for turning roadways, see ***AASHTO – "A Policy on Geometric Design of Highways and Streets" - 2011***.

C.9.d.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 - 27 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 – 34~~27~~4 Derived~~sign~~ Pavement Widths ~~of Pavements~~ for Turning Roadways for Different Design Vehicles

~~**Pavement Width (feet)**~~

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	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
Target	12	14	14	15	14	15	14	15	15	15	14	14	14

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	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
Target	17	20	20	21	20	21	20	21	21	21	20	20	20

Table Continued on Next Page

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
Target	23	26	26	27	26	27	28	27	27	27	26	26	26

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

C.9.e At Grade Intersections

C.9.e.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***.

C.9.e.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.

C.9.e.3 Median Openings

Median openings should be restricted in accordance with the requirements presented in C.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***.

C.9.e.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.

C.9.f 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.

C.9.g 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 C.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 - 27~~32~~. 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 - 27~~32~~.

Entrances to freeways should be designed in accordance with the general configurations shown below Table 3 - 26~~21~~. 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 - 26~~2~~1 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 - 27~~32~~ and 3 - 26~~21~~. The lengths obtained from Tables 3 - 27~~32~~ and 3 - 26~~21~~ should be adjusted for grade by using the ratios in Table 3 - 25~~120~~.

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***, should generally be considered as minimum criteria.

C.9.h Clear Zone

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***.

C.10 Other Design Factors

C.10.a 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.

C.10.a.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. ~~Provisions for pedestrian traffic outside of the road right of way should be considered.~~

The overall objective is to provide a safe, ~~secure,~~ continuous, convenient, and comfortable trip ~~continuity and access environment~~ for pedestrian traffic.

C.10.a.2 Accessibility Requirements

Pedestrian facilities, such as ~~walkways and~~ sidewalks, shared use paths and transit boarding and alighting areas shall be designed to accommodate people with disabilities~~physically disabled persons whose mobility is dependent on wheelchairs and other devices~~. In addition to the design criteria provided in this Manual~~chapter~~, the ***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 ***2017~~2~~ Florida Building Code – Accessibility, 6th Edition~~Accessibility Code for Building Construction~~*** as required by ***Rule Chapter 61G20-4.002, Florida Administrative Code*** impose additional requirements for the design and construction of pedestrian facilities.

C.10.a.3 Sidewalks

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 width for sidewalks is covered in ***Chapter 8 – Pedestrian Facilities*** and Section C.7.d of this chapter. To ensure compliance with federal and state accessibility requirements:

- 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.

Sidewalks 5 feet wide or wider will provide for two adults to walk comfortably side by side.

C.10.a.4 Curb Ramps

In areas with sidewalks, curb ramps must be incorporated at locations where crosswalks adjoin the sidewalks. The basic curb ramp type and design application depends on the geometric characteristics of the intersection or other crossing location.

Typical curb ramp width shall be a minimum of 4 feet with 1:10 curb transitions on each side when pedestrians must walk across the ramp. Ramp slopes shall not exceed 1:12~~0~~ and shall have a firm, stable, slip resistant surface texture. Ramp widths equal to crosswalk widths are encouraged.

Curb ramps at marked crossings shall be wholly contained within the crosswalk markings excluding any flared sides.

If diagonal ramps must be used, any returned curbs or other well-defined edges shall be parallel to the pedestrian flow. The bottom of diagonal curb ramps shall have 48-inch minimum clear space within the crosswalk. Curb ramps whose sides have returned curbs provide useful directional cues where they are aligned with the pedestrian street crossing and are protected from cross travel by landscaping or street, street furniture, or railings.

It is important for persons using the sidewalk that the location of the ramps be as uniform as possible. Detectable warnings are required at all curb ramps and flush transitions where sidewalks or shared use paths meet a roadway.

The Department's ***Standard Plans, Index 522-002*** ~~***Design Standards, Index 304***,~~ provides additional information on the design of accessible sidewalks and shared use paths. Designers should keep in mind there are many variables involved, possibly requiring each street intersection to have a unique design.

Two ramps per corner are preferred to minimize the problems with entry angle and to decrease the delay to pedestrians entering and exiting the roadway.

C.10.a.5 Additional Considerations

For additional information on pedestrian facilities design, including physical separation from the roadway, over- and underpasses, pedestrian crossings, traffic control, sight distance and lighting, refer to ***Chapter 8 – Pedestrian Facilities***.

C.10.b 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 ~~and gutter~~ 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***.

C.10.c Bridge Design Loadings

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

C.10.d Dead End Streets and Cul-de-Sacs

The end of a dead end street should permit travel return with a turn around 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***.

C.10.e 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 ~~(Table 3 - 15)~~ 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~~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*** and ***Rule Chapter 14-20.003, Florida Administrative Code*** and ***Rule Chapter 14-20.0032, F.A.C***.

C.10.f 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***.

C.11 Reconstruction

C.11.a 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.

C.11.b 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.

C.11.c 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.

C.12 Design Exceptions

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

C.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 3-17~~0~~ for lane widths for very low volume roads.

C.13.a 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.

C.13.b 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).

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CHAPTER 4

ROADSIDE DESIGN

TABLES

FIGURES

CHAPTER 4

ROADSIDE DESIGN

A INTRODUCTION

This chapter presents guidelines and standards for roadside designs intended to reduce the likelihood and/or consequences of roadside crashes. Due to the variety of causative factors, the designer should review crash reports for vehicles leaving the traveled way at any location. On average, lane departure crashes in Florida represent approximately 1/3 of all crashes and almost 50% of all highway fatalities. ~~Between 2011 and 2015, lane departure crashes in Florida represented approximately 35 percent of all crashes and approximately 44 percent of all highway fatalities.~~ Construction and maintenance of safe medians and roadsides are of vital importance in the development of safe streets and highways.~~ ~~ More information on lane departure crashes in Florida can be found in the Department's ***Florida Strategic Highway Safety Plan***.

Many of the standards presented in ***Chapter 3 – Geometric Design*** are predicated to a large extent upon reducing the probability of vehicles leaving the proper travel path. Other standards in that chapter are directed toward a reduction in the likelihood and/or consequences of crashes by vehicles leaving the roadway~~.~~, such as shoulders and medians. ~~These standards contain requirements for the design of shoulders, medians, and roadsides including requirements for the use of longitudinal barriers.~~ The d~~D~~esign of the roadside beyond the shoulder should also be considered and conducted as an integral part of the total highway design.

~~Due to the variety of causative factors, the designer should consider a vehicle leaving the traveled way at any location. Design of the roadside should be based upon reducing the consequences to errant vehicles and their occupants.~~

The general objective of roadside design is to provide an environment that will reduce the likelihood and/or consequences of crashes by vehicles that have left the traveled way. The achievement of this general objective will be aided by the following:

- Roadside areas adequate to allow reasonable space and time for a driver to regain or retain control of the vehicle and stop or return to the traveled way safely.
- Shoulders, medians, and roadsides that may be traversed safely without vehicle vaulting or overturning.

- Location of roadside fixed objects and hazards as far from the travel lane as is economically feasible.
- Roadsides that accommodate necessary maintenance vehicles, emergency maneuvers and emergency parking.
- Provide adequate shielding of hazards where appropriate and compatible with vehicle speeds and other design variables.

Prior to any other consideration, the designer should, in order of preference, attempt to:

1. Eliminate the hazard

a. Remove the hazard

b. Redesign the hazard so it can be safely traversed

c. Relocate the hazard outside the clear zone

2. Make the hazard crashworthy

3. Shield the hazard with a longitudinal barrier or crash cushion.

4. Delineate the hazard and leave the hazard unshielded. This treatment is taken only when the barrier or crash cushion is more hazardous than the hazard. See Section E.5 for information on making this determination.

This chapter contains standards and general guidelines for ~~particular~~ situations encountered in roadside design due to the variety and complexity of possible situations encountered. In addressing roadside hazards, the designer should utilize the following as basic guidelines to develop a safe roadside design.

B ROADSIDE TOPOGRAPHY AND DRAINAGE FEATURES ~~POLICY~~

B.1 Roadside Slopes, Clear Zone and Lateral Offset

Providing a sufficient amount of recoverable slope or clear zone adjacent to the roadway, free of obstacles and hazards provides an opportunity for an errant vehicle to safely recover. Minimum standards for roadside slopes, clear zone and lateral offsets to hazards are provided as follows.

B.1.a Roadside Slopes and Clear Zone

The slopes of all roadsides should be as flat as possible to allow for safe traversal by out of control vehicles. A slope of 1:4 or flatter should be used, desirably 1:6 or flatter. The transition between the shoulder and adjacent side slope should be rounded and free from discontinuities. A slope as steep as 1:3 may be used within the clear zone if the clear zone width is adjusted to provide a clear runout area as described below. If sufficient right of way exists, use flatter side slopes on the outside of horizontal curves.

Clear zone is the unobstructed, traversable area beyond the edge of the traveled way for the recovery of errant vehicles. The clear zone includes shoulders and bicycle lanes. The clear zone must be free of aboveground fixed objects, water bodies and non-traversable or critical slopes. Clear zone width requirements are dependent on AADT, design speed, and roadside slope conditions. With regard to the ability of an errant vehicle to traverse a roadside slope, slopes are classified as follows:

1. Recoverable Slope – Traversable Slope 1:4 or flatter. Motorists who encroach on recoverable foreslopes generally can stop their vehicles or slow them enough to return to the roadway safely.
2. Non-Recoverable Slope – Traversable Slope steeper than 1:4 and flatter than 1:3. Non-recoverable foreslopes are traversable but most vehicles will not be able to stop or return to the roadway easily. Vehicles on such slopes typically can be expected to reach the bottom.
3. Critical Slope – Non-Traversable Slope steeper than 1:3. A critical foreslope is one on which an errant vehicle has a higher propensity to overturn.

Clear zone widths for recoverable foreslopes 1V:4H and flatter are provided in Table 4 – 1 Minimum Width of Clear Zone. Clear zone is applied as shown in Figures 4 – 1 Clear Zone Plan View and 4 – 2 Basic Clear Zone Concept.

On non-recoverable slopes steeper than 1:4 and flatter than 1:3, a high percentage of encroaching vehicles will reach the toe of these slopes. Therefore, the clear zone distance cannot logically end at the toe of a non-recoverable slope. When such non-recoverable slopes are present within the clear zone width provided in Table 4 – 1, additional clear zone width is required. The minimum amount of additional width provided must equal the width of the non-recoverable slope with no less than 10 feet of recoverable slope provided at the toe of the non-recoverable slope. See Figure 4 – 3 Adjusted Clear Zone Concept.

When clear zone requirements cannot be met, see ***Sections C, D*** and ***E*** for requirements for roadside barriers and other treatments for safe roadside design. In addition, the ~~Department's~~ ~~***Plans Preparation Manual***~~ ~~),~~ ***AASHTO Roadside Design Guide (2011)***, and ***AASHTO Guidelines for Geometric Design of Very Low Volume Local Roads (ADT ≤ 400) (2001)*** may be referenced for a more thorough discussion of roadside design.

Table 4 – 1 Minimum Width of Clear Zone (feet)[1]

Design Speed mph	AADT ≥ 1500			AADT < 1500[1, 2]		
	Travel Lanes & Multilane Ramps		Aux Lanes and Single Lane Ramps	Travel Lanes & Multilane Ramps		Aux Lanes and Single Lane Ramps
	1V:6H or flatter	1V:5H to 1V:4H	1V:4H or flatter	1V:6H or flatter	1V:5H to 1V:4H	1V:4H or flatter
≤ 40	14	16	10	10[2]	12[2]	10[2]
45 – 50	20	24	14	14	16	14
55	22	26	18	16	20	14
60	30	30[3]	24	20	26	18
65 – 70	30	30[3]	24	24	28	18

1. Clear Zone for roads functionally classified as Local Roads with a design AADT ≤ 400 vehicles per day:
   a. A clear zone of 6 feet or more in width must be provided if it can be done so with minimum social/environmental impacts.
   b. Where constraints of cost, terrain, right of way, or potential social/environmental impacts make the provision of a 6 feet clear zone impractical, clear zones less than 6 feet in width may be used, including designs with 0 feet clear zone.
   c. In all cases, clear zone must be tailored to site-specific conditions, considering cost-effectiveness and safety tradeoffs. The use of adjustable clear zone widths, such as wider clear zone dimensions at sharp horizontal curves where there is a history of run-off-road crashes, or where there is evidence of vehicle encroachments such as scarring of trees or utility poles, may be appropriate. Lesser values of clear zone width may be appropriate on tangent sections of the same roadway.
   d. Other factors for consideration in analyzing the need for providing clear zones include the crash history, the expectation for future traffic volume growth on the facility, and the presence of vehicles wider than 8.5 feet and vehicles with wide loads, such as farm equipment.
2. May be reduced to 7 feet for a design AADT < 750 vehicles per day.
3. Greater clear zone widths provide additional safety for higher speed and volume roads. See Section 3.1 of the ***AASHTO Roadside Design Guide*** ***(2011)*** for further information.

1. Measured from the edge of the traveled way.

Source: Table 3 – 1, Suggested Clear Zone Distances in Feet from the Edge of the Travel Lane, 2011 AASHTO Roadside Design Guide.

~~The roadside, which includes the median, shall be considered as the total environment adjacent to the roadway. The design of the roadside shall be considered as an integral part of the total highway design.~~

Figure 4 – 1 Clear Zone Plan View

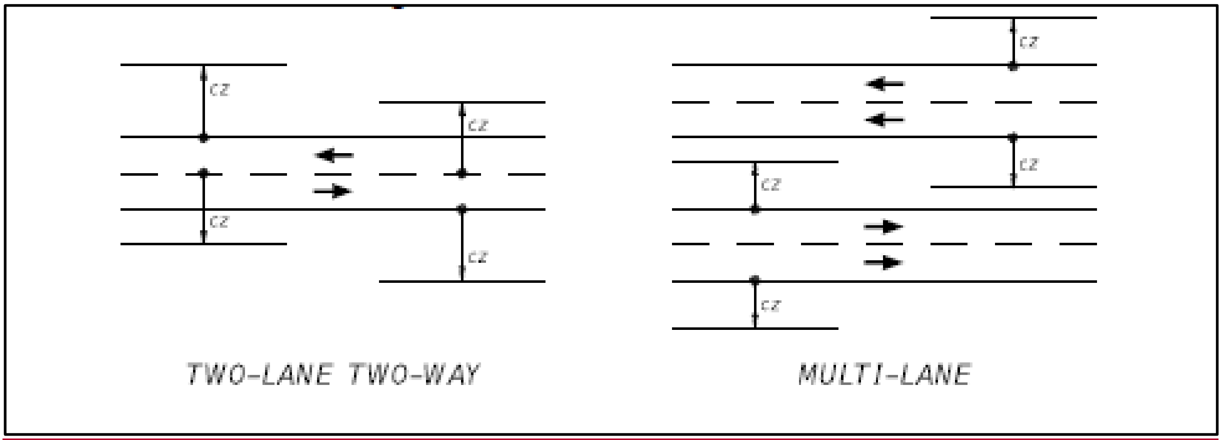


Figure 4 – 2 Basic Clear Zone Concept

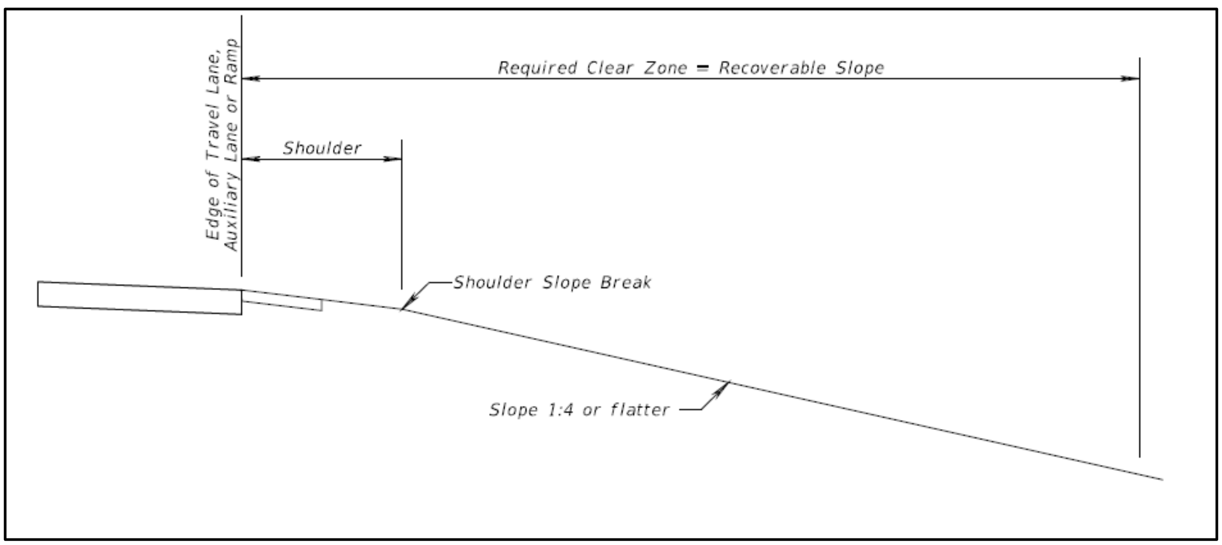
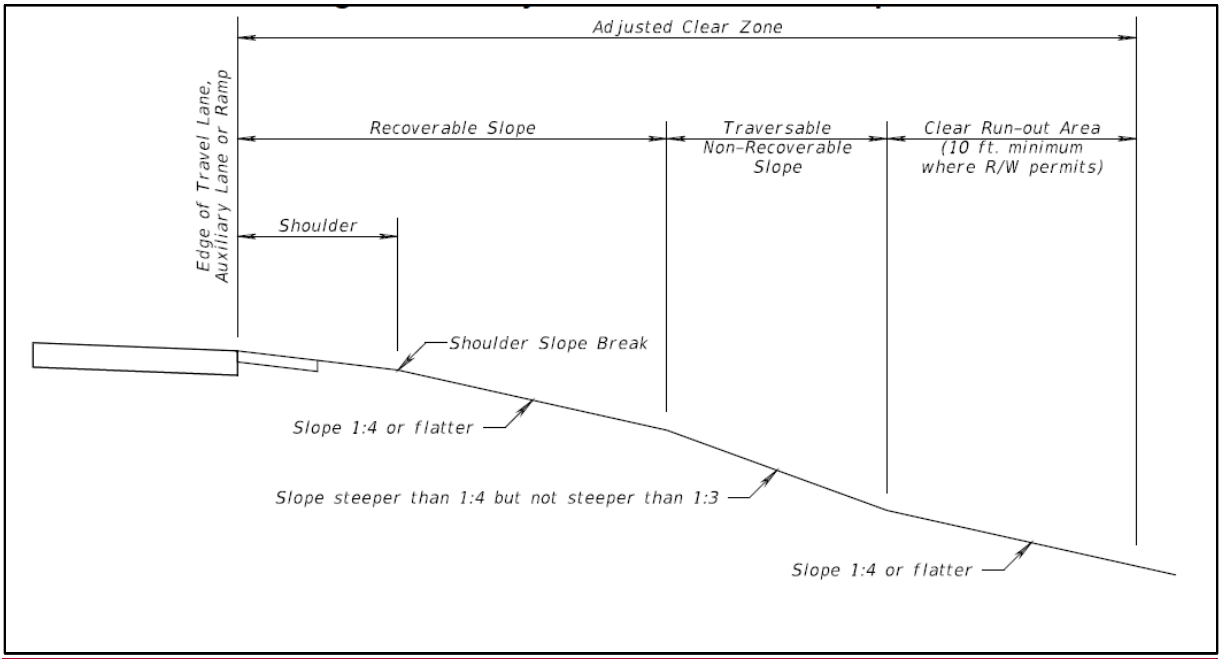
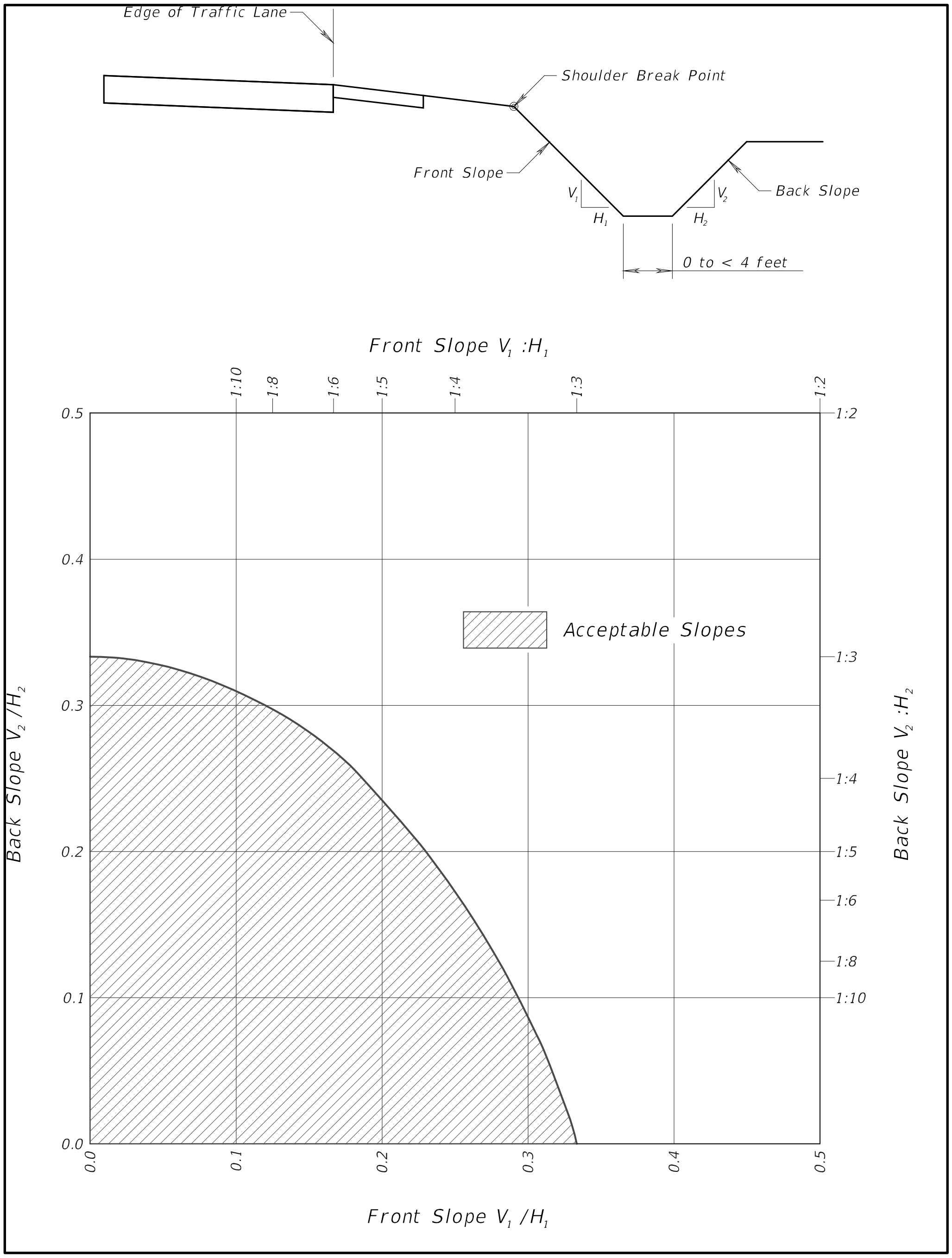


Figure 4 – 3 Adjusted Clear Zone Concept



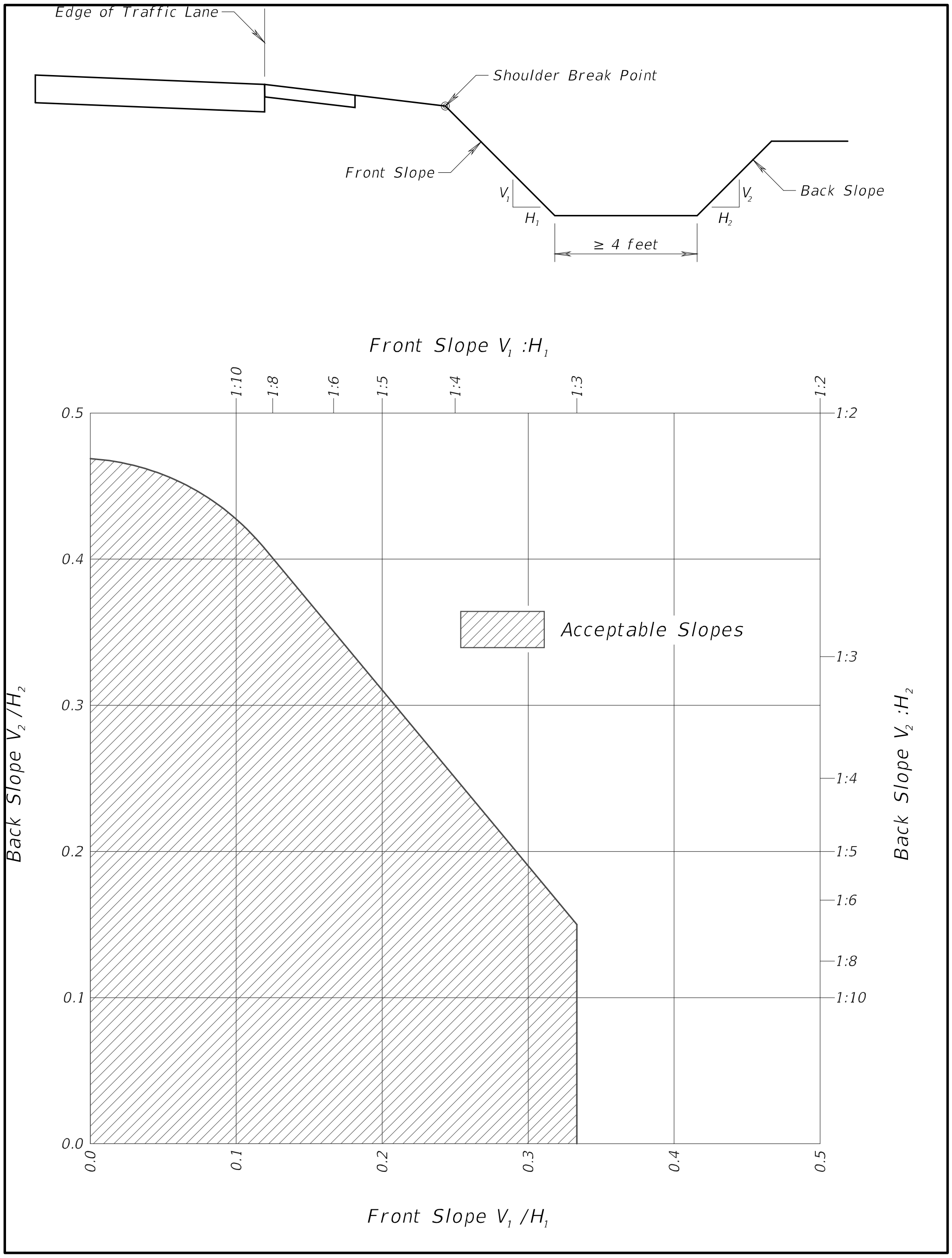
Roadside ditches may be included within the clear zone if properly designed to be traversable. Acceptable cross section slope criteria for roadside ditches within the clear zone is provided in Figure 4 – 4 Roadside Ditches – Bottom Width 0 to < 4 Feet and Figure 4 – 5 Roadside Ditches – Bottom Width ≥ 4 Feet. These roadside ditch configurations are considered traversable.

Figure 4 – 4 Roadside Ditches – Bottom Width 0 to < 4 Feet



Source: Figure 3 – 6, 2011 AASHTO Roadside Design Guide.

Figure 4 – 5 Roadside Ditches – Bottom Width ≥ 4 Feet



Source: Figure 3 – 6, 2011 AASHTO Roadside Design Guide.

B.1.b Lateral Offset

Lateral offset is the lateral distance from a specified point on the roadway such as the edge of traveled way or face of curb, to a roadside feature or above ground object that is more than 4 inches above grade. Lateral offset requirements apply to all roadways. The requirements for various objects or features are based on:

- Design speed,
- Location; i.e. rural areas or within urban boundary,
- Flush shoulder or with curb,
- Traffic volumes, and
- Lane type; e.g. travel lanes, auxiliary lanes, and ramps.

Lateral Offset requirements are provided in Table 4 – 2 Lateral Offset.

Flush shoulder roadways typically have sufficient right of way to provide the required clear zone widths. Therefore, lateral offset requirements for these type roadways are based on providing the clear zone widths provided in Table 4 – 1.

On urban curbed roadways with design speeds ≤ 45 mph, lateral offsets based on Table 4 – 1 clear zone requirements should be provided where practical. However, these urban low speed roads are typically located in areas where right of way is restricted (characterized by more dense abutting development, presence of parking, closer spaced intersections and accesses to property, and more bicyclists and pedestrians). The available right of way is typically insufficient to provide the required clear zone widths. Therefore, lateral offset requirements for above ground objects on these roadways are based on offsets needed for normal operation and not on maintaining a clear roadside for errant vehicles.

Table 4 – 2 Lateral Offset (feet)

Roadside Feature	Urban Curbed Roadways Design Speed ≤ 45 (mph)	All Other
Above Ground Objects[1]	4 ft. from Face of Curb[2]	Clear Zone Width
Drop Off Hazards[3]	Clear Zone Width	Clear Zone Width
Water Bodies	Clear Zone Width	Clear Zone Width
Canal Hazards	See Section B.2.c	See Section B.2.c

1. Above ground objects are anything greater than 4 inches in height and are firm and unyielding or do not meet crashworthy or breakaway criteria. For urban curbed areas ≤ 45 mph this also includes crashworthy or breakaway objects except those necessary for the safe operation of the roadway.
2. May be reduced to 1.5 ft. from Face of Curb on roads functionally classified as Local Streets and on all roads where the 4 ft. minimum offset cannot be reasonably obtained and other alternatives are deemed impractical.
3. Drop off hazards are:
   a. Any vertical faced structure with a drop off (e.g. retaining wall, wing-wall, etc.) located within the Clear Zone.
   b. Slopes steeper than 1:3 located within the Clear Zone.
   c. Drop-offs with significant crash history.

B.2 Drainage Features

Drainage design is an important aspect of the long-term performance of a roadway, and to achieve an effective design, drainage features are necessary in close proximately to travel lanes. These features include ditches, curbs, and drainage structures (e.g. transverse/parallel pipes, culverts, endwalls, wingwalls, and inlets). The placement of these features is to be evaluated as part of roadside safety design. Refer to ***Chapter 20 – Drainage*** for information regarding proper hydraulic design.

When evaluating the design of roadside topography and drainage features, consider the future maintenance implications of the facility. Routine maintenance or repairs needed to ensure the continued function of the roadway slopes or drainage may lead to long-term expenses and activities, which disrupts traffic flow and exposes maintenance personnel to traffic conditions.

B.2.a Roadside Ditches

Minimum standards for side slopes and bottom widths of roadside ditches and channels within the clear zone are provided in Section B.1.a.

B.2.b Drainage Structures

Drainage structures and their associated end treatments located along the roadside should be implemented using either a traversable design or located outside the required clear zone. The various drainage inlets and pipe end treatments needed for an efficient drainage design typically contain curb inlets, ditch bottom inlets, endwalls, wingwalls, headwalls, flared end sections and/or mitered end sections. If not adequately designed or properly located, these features can create hazardous conditions (e.g. abrupt deceleration or rollovers) for vehicles. For detailed background information concerning traversable designs, refer to the ***AASHTO Roadside Design Guide***.

Standard details for drainage structures and end treatments commonly used in Florida are provided the in the Department's ~~***Design***~~ ***Standards Plans Index 425, 430,*** and ***436*** ~~***200***~~ ***Series***. Drainage features shown in the Department's ~~***Design***~~ ***Standard Plans*** have the potential for conflict with a vehicle either departing the roadway or within a commonly traversed section of a roadway. The D~~d~~epartment's ***Drainage Manual*** identifies those standard drainage structures which are acceptable for use within the clear zone.

B.2.c Canals and Water Bodies

~~Roadside canals or other bodies of water close to the roadway should be eliminated wherever feasible. A canal is defined as an open ditch parallel to the roadway for a minimum distance of 1000 ft. and with a seasonal water depth in excess of 3 ft. for extended periods of time (24 hours or more).~~

~~Where roadside bodies of water (with seasonal water depth in excess of 3 feet for 24 hours or longer) lie within the roadside clear zone, they shall be shielded using guardrail or another longitudinal barrier.~~

~~For rural and urban flush shoulder highways, the distance from the outside edge of the through travel lane to the top of the canal side slope nearest the road will be no less than 60 ft. for highways with design speeds of 50 mph or greater. For highways with design speeds less than 50 mph this minimum distance shall not be less than 50 ft. for rural and urban flush shoulder highways or 40 ft. for urban curb or curb and gutter highways. When new canal or roadway alignment is required, distances greater than those above should be provided, if possible, to accommodate possible future improvements to the roadway (widening, etc.). If the minimum standards for canal hazards cannot be met, then shielding should be considered~~

Roadside canals and other bodies of water close to the roadway should be eliminated wherever feasible. When not feasible, they should be located outside of the clear zone as shown in Table 4 – 1 Minimum Width of Clear Zone. If the body of water meets the definition of a canal hazard, additional lateral offset is required for arterial and collector roadways.

A canal hazard is defined as an open ditch parallel to the roadway for a minimum distance of 1,000 feet and with seasonal water depth more than 3 feet for extended periods of time (24 hours or more).

~~Roadside canals and other bodies of water close to the roadway should be eliminated wherever feasible. Roadside water bodies that do not meet the definition of a canal hazard shall be located outside the clear zone as shown in Table 4 – 1. For canal hazards on arterial or collector roadways, additional lateral offset is required. A canal hazard is defined as an open ditch parallel to the roadway for a minimum distance of 1,000 feet and with a seasonal water depth in excess of 3 feet for extended periods of time (24 hours or more).~~

Canal hazard lateral offset is the distance from the edge of travel lane, auxiliary lane or ramp to the top of the canal side slope nearest the road. Minimum required lateral offset distances are as follows: ~~Rural and Urban~~

- Not less than 60 feet for flush shoulder and curbed roadways with design speeds of 50 mph or greater.
- Not less than 50 feet for flush shoulder roadways with design speeds of 45 mph or less.
- Not less than 40 feet for curbed roadways with design speeds of 45 mph or less.

See also Figure 4 – 6 Minimum Offsets for Canal Hazards (Flush Shoulders) and Figure 4 – 7 Minimum Offsets for Canal Hazards (Curb and Curb and Gutter). On new alignments and/or for new canals, greater distances should be provided to accommodate future widening of the roadway.

On fill sections, a flat berm (maximum 1:10 slope) no less than 20 feet in width between the toe of the roadway front slope and the top of the canal side slope nearest the roadway should be provided.

When the slope between the roadway and the "extended period of time" water surface is 1:6 or flatter, the minimum distance can be measured from the edge of the travel lane, auxiliary lane, or ramp to the "extended period of time" water surface. ~~and~~A~~a~~ berm is not required.

On sections with ditch cuts, a minimum of 20 feet between the toe of the front slope and the top of the canal side slope nearest the roadway should be provided.

When the required minimum lateral offset cannot be met, the canal hazard shall be shielded with a crashworthy roadside barrier. Barriers shall be located as far from the travel way as practical. When shielding canal hazards the barrier shall be located outside the clear zone where possible. Guardrail shall be located no closer than 6 feet from the canal front slope and high tension cable barrier shall be no closer than 15 feet from the canal front slope.

Figure 4 – 6 Minimum Offsets for Canal Hazards
~~Rural and Urban~~ (Flush Shoulders)

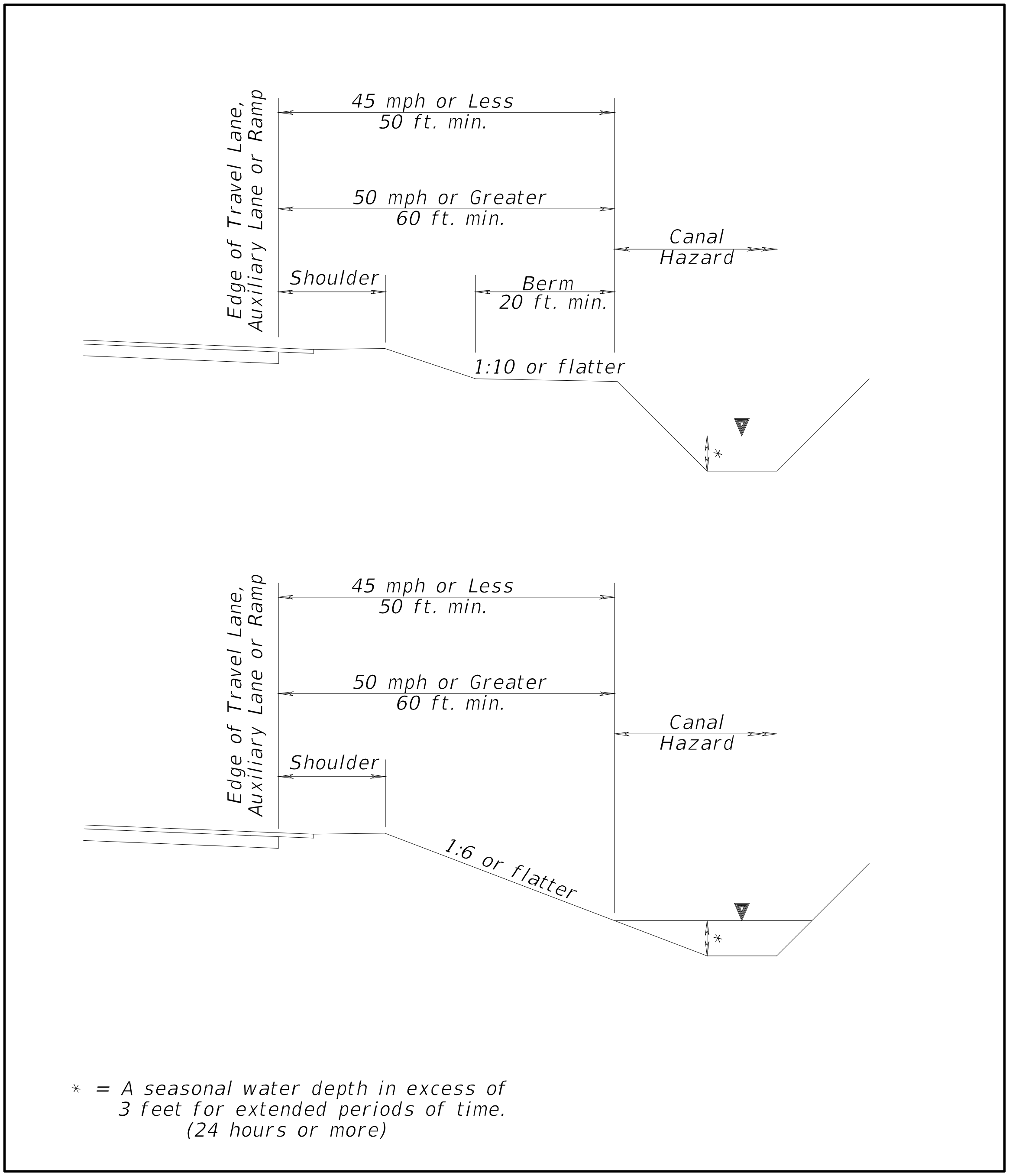
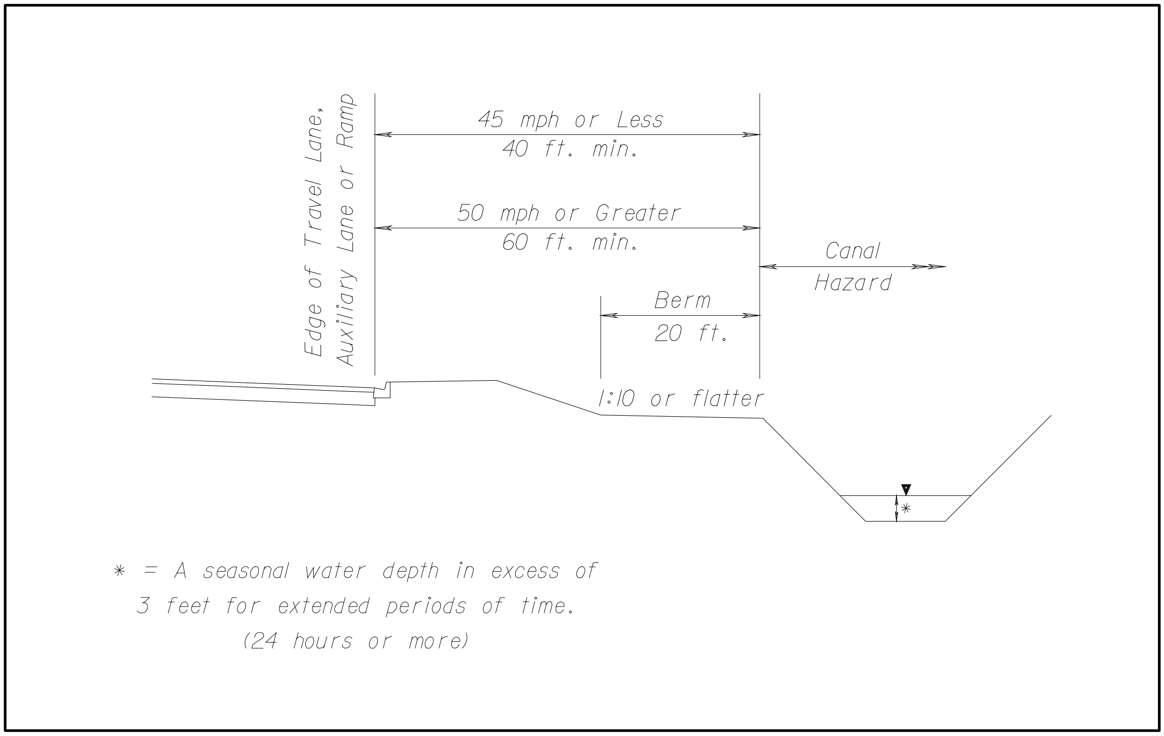


Figure 4 – 7 Minimum Offsets for Canal Hazards
~~Urban~~ (Curbed ~~or Curb and Gutter~~)



B.2.d Curb

Curbs with closed drainage systems are typically used in urban areas to minimize the amount of right of way needed. Curbs also provide a tangible definition of the roadway limits and delineation of access points. These functions are important in urban areas because of the following typical characteristics:

- Low design speed (Design Speed ≤ 45 mph);
- Dense abutting development;
- Closely spaced intersections and accesses to property;
- Higher number of motorized vehicles, bicyclists and pedestrian volumes, and;
- Restricted right of way.

Chapter 3 – Geometric Design provides criteria on the use of curbs. It should be noted that curbs have no redirectional capabilities except at very low speeds; less than the lowest design speeds typically used for urban streets. Therefore, curbs are not considered to be effective in shielding a hazard and are not to be used to reduce lateral offset requirements.

The Department's ***~~Design~~*** ***Standard Pans, Index 520-001*** ***~~300~~*** provides standard details for curb shapes commonly used in Florida. Typical applications for urban roadways include Type E and Type F curbs. Both curb types have a sloped face; however, the Type E has a flatter face to allow vehicles to traverse it more easily. Shoulder gutter is also frequently used along roadway fill sections and bridge approaches to prevent excessive runoff down embankment slopes. The Department's ***Drainage Manual*** may be referenced for direction on the use of shoulder gutter.

Curbs types such as Type E (height 5" or less with a sloping face equal to or flatter than the Type E) may be used in the following cases on high speed roadways. The face of the curb shall be placed no closer to the edge of the traveled way than the required shoulder width.

- High speed multilane divided highways with design speeds of 55 mph and less. For examples see the Department's ***Design Manual, Chapter 210 Arterials and Collectors.***
- Directional Median Openings. For examples see the Department's ***Design Manual, Chapter 212 Intersections*** ***~~Standard Plans Index 527~~***.
- Transit Stops (harmonize with flush shoulder accessible transit stops).

C ROADSIDE SAFETY FEATURES AND CRASH TEST CRITERIA~~OBJECTIVES~~

While a traversable and unobstructed roadside is highly desirable from a safety standpoint, some appurtenances near the traveled way are necessary. Man-made fixed objects that frequently occupy road rights-of-way include traffic signs, traffic signals, roadway lighting, railroad warning devices, intelligent transportation systems (ITS), utility poles, mailboxes. Other features include safety hardware such as barriers, end treatments and crash cushions which are often necessary to shield errant motorists from a variety of roadside hazards.

These features are in addition to trees and other vegetation often present, either naturally occurring or as part of landscaping. Applicable criteria for each of these features is presented in the following sections. Certain features are required to meet specific crash test criteria involving full scale crash testing.

C.1 Crash Test Criteria

Crash test criteria for roadside safety features has been in existence since 1962, but has changed over time as the vehicle fleet changes, and crash characteristics and hardware performance becomes better understood. ***NCHRP Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features,*** published in 1993, has been the accepted criteria for safety hardware device testing for many years.

More recently, the ***AASHTO Manual for Assessing Safety Hardware (MASH)*** was published and has superseded ***NCHRP Report 350*** as the most current criteria. To allow adequate time for the testing and development of features under MASH criteria, safety hardware installed on new and reconstruction projects shall meet ***NCHRP Report 350*** crash test criteria as a minimum. For projects on the National Highway System, a schedule has been established for implementing requirements for devices meeting MASH criteria. For more information see FHWA's Web Site for ***Roadway Departure Safety***. New and reconstruction projects not on the National Highway System are not required to conform to this implementation schedule, but should comply to the extent practical.

The Department maintains standard details, specifications and approved products for all types of roadside devices commonly used in Florida that meet the required crash test criteria, and are acceptable for use on all public roadways. Nonproprietary, standardized devices are detailed in the Departments ~~***Design Standards***~~ ***Standard Plans, Indexes 521, 536, and 544 Series***. Proprietary products are included on the Department's ***Approved Product List (APL).*** These devices address the majority of roadside needs for all roads in Florida. The most current version of the ~~***Design***~~ ***Standard Plans*** and ***APL*** should be used as the Department maintains and updates these publications as necessary to comply with required implementation dates for changes in crash test criteria.

For cases where a device may be needed that is not covered by the Department's standards and approved products, the Federal Highway Administration (FHWA) maintains lists of eligible crashworthy devices, which can be found on their website for ***Roadway Departure Safety***. In addition, the AASHTO-Associated General Contractors of America (AGC}-American Road and Transportation Builders Association (ARTBA) Joint Committee Task Force 13 report, *A Guide to Standardized Highway Barrier Hardware*, provides engineering drawings for a multitude of barrier components and systems.

The criteria for crash testing specified in ***NCHRP Report 350*** and ***AASHTO MASH*** provides six Test Levels (TL-1 thru TL-6) for the evaluation of roadside hardware suitability. A test level is defined by impact speed and angle of approach, and the type of test vehicle. Test vehicles range in size from a small car to a loaded tractor trailer truck. Each Test Level provides an increasing level of service in ascending numerical order.

Tables 4 – 3 Test Levels for Barriers, End Terminals, Crash Cushions and 4 – 4 Test Levels for Breakaway Devices, Work Zone Traffic Control Devices summarize the vehicle types, vehicle mass, test speeds and impact angles used in testing for each test level. Tables 4 – 3 and 4 – 4 also show the differences in vehicle mass between MASH and ***NCHRP Report 350*** criteria for the small car, pickup and single unit truck test vehicles.

In addition to differences in vehicle mass, MASH test criteria incorporated several other changes that differ from ***NCHRP Report 350***. For additional information on crash test criteria, refer to the ***AASHTO MASH***, ***NCHRP Report 350***, the ***AASHTO Roadside Design Guide***, and the FHWA web site for ***Roadway Departure Safety***.

Table 4 – 3 Test Levels for Barriers, End Terminals, Crash Cushions

Test Level	Test Vehicle Type	Vehicle Designation and Mass		Test Conditions MASH	
		NCHRP 350 (lbs.)	**MASH** (lbs.)	**Impact Speed** (mph)	**Impact Angle (for Barriers)** (degrees)
1	Passenger Car Pickup Truck	820C 1800 2000P 4400	1100C 2420 2270P 5000	31 31	25 25
2	Passenger Car Pickup Truck	820C 1800 2000P 4400	1100C 2420 2270P 5000	44 44	25 25
3	Passenger Car Pickup Truck	820C 1800 2000P 4400	1100C 2420 2270P 5000	62 62	25 25
4	Passenger Car Pickup Truck Single-Unit Truck	820C 1800 2000P 4400 8000S 17640	1100C 2420 2270P 5000 10000S 22000	62 62 56	25 25 15
5	Passenger Car Pickup Truck Tractor-Van Trailer	820C 1800 2000P 4400 36000V 79300	1100C 2420 2270P 5000 36000V 79300	62 62 50	25 25 15
6	Passenger Car Pickup Truck Tractor-Tank Trailer	820C 1800 2000P 4400 36000V 79300	1100C 2420 2270P 5000 36000V 79300	62 62 50	25 25 15
Note: Test Levels 1, 2 and 3 apply to end terminals and crash cushions, while all 6 Test Levels apply to barriers.					

Table 4 – 4 Test Levels for Breakaway Devices, Work Zone Traffic Control Devices

Test Level	**Feature**	**Test Vehicle Type**	**Vehicle Designation and Mass**		**Impact Speeds**		**Impact Angle** (degrees)
			NCHRP 350 (lbs.)	**MASH** (lbs.)	**Low Speed** (mph)	**High Speed** (mph)	
2	Support Structures and Work Zone Traffic Control Devices	Passenger Car Pickup Truck	820C 1800 Not Required	1100C 2420 2270P 5000	19 19	44 44	0 – 20 0 – 20
	Breakaway Utility Poles	Passenger Car Pickup Truck	820C 1800 Not Required	1100C 2420 2270P 5000	31 31	44 44	0 – 20 0 – 20
3	Support Structures and Work Zone Traffic Control Devices	Passenger Car Pickup Truck	820C 1800 Not Required	1100C 2420 2270P 5000	19 19	62 62	0 – 20 0 – 20
	Breakaway Utility Poles	Passenger Car Pickup Truck	820C 1800 Not Required	1100C 2420 2270P 5000	31 31	62 62	0 – 20 0 – 20
Note: Criteria for Test Levels 2 and 3 are provided for support structures, work zone traffic control devices and breakaway utility poles. Test Level 3 is the basic test level used for most devices.							

As noted in Tables 4 – 3 and 4 – 4, Test Levels 1 through 3 are limited to passenger vehicles while Test Levels 4 through 6 incorporate heavy trucks. The test speeds and impact angles used for testing represent approximately 92.5% of real word crashes. As implied by the information in Tables 4 – 3 and 4 – 4:

1. Test Level 1 devices should be used only on facilities with design speeds 30 mph and less.

2. Test Level 2 devices should be used only on facilities with design speeds 45 mph and less.

3. Test Level 3 through Test Level 6 devices are considered acceptable for all design speeds.

4. Test Level 3 devices are generally considered acceptable for facilities of all types and most roadside conditions.

5. Test Levels 4 through 6 should be considered on facilities with high volumes of heavy trucks and/or where penetration beyond the barrier would result in high risk to the public or surrounding facilities.

For additional information regarding appropriate application of Test Levels refer to the ***AASHTO Roadside Design Guide***.

C.2 Safety Hardware Upgrades

On new construction and reconstruction projects existing obsolete safety hardware shall be upgraded or replaced with hardware meeting crash test criteria as described above.

For existing roadways, highway agencies should upgrade existing highway safety hardware to comply with current crash test criteria either when it becomes damaged beyond repair, or when an individual agency's maintenance policies require an upgrade to the safety hardware.

The Department's ***Design ~~Plans Preparation~~Manual, Chapter 215 Roadside Safety*** provides a list of considerations when investigating the need for upgrading barriers and other hardware. The Department's Design Standards provide standard details for transitioning new barriers to existing barriers. The ***AASHTO Roadside Design Guide*** also provides guidelines for upgrading hardware.

~~General objectives to be followed in roadside design are to provide an environment that will reduce the likelihood and/or consequences of crashes by vehicles that have left the traveled way. The achievement of this general objective will be aided by the following:~~

- ~~Roadside areas adequate to allow reasonable space and time for a driver to regain or retain control of the vehicle and stop or return to the traveled way safely.~~

- ~~Shoulders, medians, and roadsides that may be traversed safely without vehicle vaulting or overturning.~~

- ~~Location of roadside fixed objects and hazards as far from the travel lane as is economically feasible.~~
- ~~Roadsides that accommodate necessary maintenance vehicles, emergency maneuvers and emergency parking.~~
- ~~Protection of pedestrians, workers, or other persons subjected to the hazard of errant vehicles.~~
- ~~Adequate protective devices (where hazards are unavoidable) compatible with vehicle speeds and other design variables.~~

D SIGNS, SIGNALS, LIGHTING SUPPORTS, UTILITY POLES, TREES AND SIMILAR ROADSIDE FEATURES ~~ROADSIDE DESIGN~~

D.1 General

This section provides criteria for traffic sign supports, signal supports, lighting supports, utility poles, trees and similar roadside features.

Generally, those roadside appurtenances and features that cannot be removed or located outside the clear zone must meet breakaway criteria to reduce impact severity. For those features located within the clear zone where it is not practical to meet breakaway criteria, shielding may be warranted and shall be considered.

D.2 Performance Requirements for Breakaway Devices

The term breakaway support refers to traffic sign, highway lighting, and other supports that are designed to yield, fracture, or separate when impacted by a vehicle. The release mechanism may be a slip plane, plastic hinge, fracture element, or combination thereof. Crash test criteria applicable to breakaway devices are presented in Section C. Additional requirements for breakaway supports are provided in the ***AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals***. For a more detailed discussion on breakaway supports, refer to the ***AASHTO Roadside Design Guide***.

See Section C for references that provide additional information and details on crash tested breakaway supports.

D.3 Sign Supports

Traffic signs and sign supports shall meet the requirements provided in the ***Manual on Uniform Traffic Control Devices (MUTCD)*** as stated in ***Chapter 18 – Signing and Marking***. The ***MUTCD*** requires all sign supports within the clear zone to be shielded or breakaway. See Section B for clear zone requirements. Only when the use of breakaway supports is not practicable should a traffic barrier or crash cushion be used exclusively to shield sign supports. In addition, sign supports should be located where they are least likely to be hit. Where possible, signs should be placed behind existing roadside barriers beyond the design deflection distance or on existing structures.

The Department's ***Standard Plans, Index 700 Series*** ~~***Design Standards*** Index 11000 Series~~ provides details for breakaway supports for single and multi-post ground mounted signs that are acceptable for use within the clear zone. The most current version of these ~~***Design***~~ ***Standard Plans*** details should be used as the Department maintains and updates these details as necessary to comply with required implementation dates for changes in crash test criteria.

Overhead signs and cantilever signs require relatively large size support systems. The potential safety consequences of these systems falling necessitate a fixed-base design that cannot be made breakaway. Overhead sign and cantilever sign supports therefore are required to be located outside the clear zone (Section B) or be shielded with a crashworthy barrier (Section E). Where possible, these supports should be located behind traffic barriers shielding nearby overpasses or other existing structures, or the signs should be mounted on the nearby structure. The Department's ~~***Design***~~ ***Standard Plans, Indexes 700-012***~~***11870***~~ and ***700-013***~~***11871***~~ provide details and instructions for the design of these systems.

D.4 Traffic Signal Supports

Traffic signal supports commonly used in Florida are fixed base and shall meet the required lateral offset and clear zone criteria provided in Section B. Traffic signal supports should not be located within medians. The Department's ~~***Design***~~ ***Standard Plans, Indexes 641-010, 649-010,*** *and* ***649-030***~~***1770***0 ***Series***~~ provide details and instructions for the design of traffic signal supports.

D.5 Lighting Supports

Lateral offset criteria for lighting supports depend on whether the support is breakaway or fixed base as discussed below. See ***Chapter 6 - Lighting*** for additional design criteria for lighting.

D.5.a Conventional Lighting

Supports for conventional lighting (heights up to 60 feet) shall be breakaway which are typically frangible bases (cast aluminum transformer bases), slip bases, or frangible couplings (couplers). The Department's ~~***Design***~~ ***Standard Plans, Indexes 715-001*** and ***715-002***~~***17500*** and ***17515***~~ provide further information for breakaway lighting supports which are acceptable for use. As a general rule, a breakaway lighting support will fall near the line of the path of an impacting vehicle. The mast arm usually rotates and points away from the roadway when resting on the ground. For poles located on the outside of the roadway (not in medians), this action generally results in the pole not falling into other traffic lanes. However, the designer should remain aware that these falling poles may endanger other motorists or bystanders such as pedestrians and bicyclists. The ***AASHTO Roadside Design Guide*** may be referenced for additional discussion on breakaway lighting supports.

On curbed roadways with design speeds 45 mph or less, breakaway lighting supports shall be located to meet lateral offset requirements provided in Section B, Table 4 – 2.

On flush shoulder roadways, breakaway lighting supports shall be located a minimum of 20 feet from the nearest travel lane, 14 feet from the nearest auxiliary lane or outside the clear zone provided in Section B, Table 4 – 1, whichever is less. The foreslope shall be 1:6~~6:1~~ or flatter in cases where supports are located within the clear zone.

Lighting should not be located in medians, except in conjunction with barriers that are justified for other reasons.

D.5.b High Mast Lighting

High mast or high-level lighting supports are fixed-base support systems that do not yield or break away on impact. High mast lighting supports shall be located outside the clear zone provided in Section B, Table 4 – 1. High mast lighting shall not be located in medians except in conjunction with barriers that are justified for other reasons. The Department's ~~***Design***~~ ***Standard Plans, Index 715-010~~17502~~*** provides additional information.

D.6 Utility Poles

Utility poles shall be located to meet lateral offset and clear zone requirements provided in Section B and be located as close as practical to the right of way line. They should be installed per the permitting agency's requirements~~. The location of new poles or relocated poles shall provide at least 48" minimum unobstructed sidewalk width.~~ The ***AASHTO Roadside Design Guide*** provides additional discussion and guidance on utility poles.

In accordance with ***Section 337.403, Florida Statutes***, existing utility poles must be relocated when ***unreasonably*** interfering with the "convenient, safe, or continuous use, or the maintenance, improvement, extension, or expansion" of public roads. Utility poles adjacent to road improvement projects, but not directly interfering with construction, should be considered for relocation, to the extent they can be relocated, to achieve the clear zone requirements of Table 3-12. Utility poles that cannot be relocated and will remain within the clear zone, should be approved through the exception process prescribed in ***Chapter 14 - Design Exceptions***.

D.7 Trees

Trees with a diameter greater than 4 inches measured 6 inches above grade shall be located to meet lateral offset and clear zone requirements in Section B, Tables 4 – 1 and 4 – 2. The ***AASHTO Roadside Design Guide*** provides additional discussion and guidance on trees.

D.8 Miscellaneous

D.8.a Fire Hydrants

Most fire hydrants are made of cast iron and are expected to fracture upon impact, however, crash testing meeting current criteria has not been done to verify that designs meet breakaway criteria. For this reason, fire hydrants should be located as far from the travel way as practical and preferably outside lateral offset/clear zone requirements in Section B, yet where they are still readily accessible to and usable by emergency personnel. Any portion of the hydrant not designed to break away should be within 4 inches of the ground.

D.8.b Railroad Crossing Warning Devices

See ***Chapter 7 – Rail-Highway Crossings*** for location requirements for railroad crossing warning devices.

D.8.c Mailbox Supports

Mailboxes and their location are subject to US Postal Service requirements. They are often located within the clear zone and pose a potential hazard.

However, with proper design and placement, the severity of impacts with mailboxes can be reduced. To achieve consistency, it is recommended each highway agency adopt regulations for the design and placement of mail boxes within the right of way of public highways. The ***AASHTO Roadside Design Guide*** provides a model regulation that is compatible with US Postal Service requirements.

The following requirements apply to mailbox installations on public roadways:

No mailbox will be permitted where access is obtained from a freeway or where access is otherwise prohibited by law or regulation. Mailboxes shall be located as follows:

- On the right-hand side of the roadway in the carrier's direction of travel except on one-way streets, where they may be placed on the left-hand side.
- Where a mailbox is located at a driveway entrance, it shall be placed on the far side of the driveway in the carrier's direction of travel.
- Where a mailbox is located at an intersecting road, it shall be located a minimum of 200 feet beyond the center of the intersecting road in the carrier's direction of travel. This distance may be decreased to 100 feet on very low volume roads.
- When a mailbox is installed in the vicinity of an existing guardrail, it should, when practical, be placed behind the guardrail.

The bottom of the box shall be set at a height established by the U. S. Postal Service, usually from 41 to 45 inches above the roadway surface.

On flush shoulder roadways, the roadside face of the box should be offset from the edge of the traveled way a distance no less than the greater of the following:

- 8 feet (where no paved shoulder exists and shoulder cross slope is 10 percent or flatter), or
- width of the shoulder present plus 6 to 8 inches, or
- width of a turnout specified by the jurisdiction plus 6 to 8 inches.

On very low volume flush shoulder roads with low operating speeds the offset may be reduced to 6 feet from the traveled way. ~~On very low volume roads with low operating speeds the offset may be reduced to as low as 32" when approved by the maintaining agency.~~

On curbed streets, the roadside face of the mailbox should be set back from the face of the curb at a distance of ~~between~~6 to ~~and~~8 inches. On residential streets without curbs or all-weather shoulders that carry low traffic volumes operating at low speeds, the roadside face of the mailbox should be offset between 8 inches and 12 inches behind the edge of the pavement.

Design criteria for the mailbox support structure when located within the clear zone should consist of the following:

- Mailboxes shall be of light sheet metal or plastic construction conforming to the requirements of the U. S. Postal Service. Newspaper delivery boxes shall be of light metal or plastic construction of minimum dimensions suitable for holding a newspaper.
- No more than two mailboxes may be mounted on a support structure unless crash tests have shown the support structure and mailbox arrangement to be safe. However, light-weight newspaper boxes may be mounted below the mailbox on the side of the mailbox support.
- ~~Mailbox supports should not be set in concrete unless crash tests have shown the support design to be safe.~~
- A single 4 inch by 4 inch square or 4 inch diameter wooden post; or metal post, Schedule 40, 2 inch (normal size IPS (external diameter 2-3/8 inch) (wall thickness 0.154 inches) or smaller), embedded no more than 24 inches into the ground, shall be acceptable as a mailbox support. A metal post shall not be fitted with an anchor plate, but it may have an anti-twist device that extends no more than 10 inches below the ground surface.
- Unyielding supports such as heavy metal pipes, concrete posts, brick, stone or other rigid foundation structure or encasement should be avoided.
- The post-to-box attachment details should be of sufficient strength to prevent the box from separating from the post top if the installation is struck by a vehicle. The exact support hardware dimension and design may vary, such as having a two-piece platform bracket or alternative

slot-and-hole locations. The product must result in a satisfactory attachment of the mailbox to the post, and all components must fit together properly.

- The minimum spacing between the centers of support posts should be the height of the posts above the ground line. Mailbox support designs not described in this regulation are acceptable if approved by the jurisdiction.

The Department's ~~FDOT Design~~ ***Standard Plans, Index 110-200*** and the ***AASHTO Roadside Design Guide*** provide details on hardware, supports and attachment details acceptable for mailboxes located within the clear zone which conform to the above requirements.

D.8.d Bus Benches and Shelters

See ***Chapter 3 – Geometric Design*** for location criteria for bus benches and shelters. Additional criteria are provided in ***Chapter 13 – Public Transit***.

~~The basic requirements and standards for the design of shoulders, medians, and roadsides are given in ***Chapter 3 - Geometric Design***. This includes specific requirements regarding widths, slopes, and changes in grade. General requirements for drainage facilities, utilities, transit, and pedestrian facilities are also included.~~

~~This chapter contains general guidelines for particular situations encountered in roadside design due to the variety and complexity of possible situations encountered. The designer should utilize the following as basic guidelines to develop a safe roadside design.~~

~~Prior to any other consideration, the designer should attempt to:~~

1. ~~Eliminate the hazard;~~
    a. ~~Remove the hazard,~~
    b. ~~Relocate the hazard outside of the clear zone,~~
    c. ~~Make the hazard traversable or crashworthy.~~
2. ~~Shield the hazard with a longitudinal barrier or crash cushion.~~
3. ~~Leave the hazard unshielded. This treatment is taken only when the barrier or crash cushion is more hazardous than the hazard.~~

~~The AASHTO Roadside Safety Analysis Program (RSAP) is the recommended tool for evaluating the cost effectiveness of shielding roadside hazards.~~

~~D.1 Geometric Changes~~

~~D.1.a Horizontal Curves~~

~~On horizontal curves, consideration should be given to increasing the clear zone above the minimum requirements due to the increased likelihood of vehicles leaving the traveled way. Increasing clear zone widths and decreasing roadside slopes on curves is also important since a vehicle will probably leave the traveled way at a steeper exit angle. Increasing clear zone widths on curves is also beneficial in improving the available sight distance. Proper signage should be part of every roadside design. For proper signage to inform drivers of approaching curves, refer to the MUTCD~~

~~D.1.b Vertical Curves~~

~~As a vehicle comes over the crest of a vertical curve, the driver may suddenly be presented with a situation requiring an emergency maneuver. The provision of adequate clear zones is particularly important where available stopping sight distance may not be adequate or where driver expectancy may be violated. High traffic volumes (i.e., urban areas) may result in rapidly forming traffic queues, thus tending to cause rear-end collisions. Vertical curves with inadequate stopping sight distance may be mitigated with appropriate advanced signage and other warning devices, or can be reconstructed.~~

~~D.1.c Changes in Cross Section~~

~~The provision of adequate clear zone is very important at exits, entrances, lane drops, or other changes in the roadway cross section. The exterior boundaries of the clear zone should extend well beyond any reductions in roadway width and then gradually reduce to provide design width for the new roadway cross section.~~

~~D.1.d Decision or Conflict Points~~

~~Adequate clear zones should be provided at any point of traffic merging or~~

~~conflicts, and at locations where the driver is confronted with making a decision regarding vehicle maneuvers.~~

~~D.2 Fills~~

~~Many roadways, for drainage purposes, are elevated somewhat above the surrounding terrain. Where feasible, the side slopes should not exceed a ratio of 1:4. On flatter slopes (1:6 or greater), care should be exercised to eliminate sharp changes in grade or other discontinuities.~~

~~If the side slope is steeper than 1:3, longitudinal barriers should be considered.~~

~~D.3 Cuts~~

~~A primary objective of roadside design in cut sections is to prevent conditions tending to cause rollovers or serious collisions with the cut slopes. When the material (soils) in the cut is smooth and stable, the use of an increasing backslope is a reasonable solution. The technique is also acceptable in stable rock cuts, provided that smooth fill material is utilized to affect the backslope.~~

~~The use of a rigid barrier incorporated into the cut slope is also satisfactory for rock slopes. Where the material in the cut is irregular or unstable, a longitudinal barrier offset from the cut face should be utilized.~~

~~D.4 Roadside Canals~~

~~Roadside canals or other bodies of water close to the roadway should be eliminated wherever feasible. A canal is defined as an open ditch parallel to the roadway for a minimum distance of 1000 ft. and with a seasonal water depth in excess of 3 ft. for extended periods of time (24 hours or more).~~

~~Where roadside bodies of water (with seasonal water depth in excess of 3 feet for 24 hours or longer) lie within the roadside clear zone, they shall be shielded using guardrail or another longitudinal barrier.~~

~~For rural and urban flush shoulder highways, the distance from the outside edge of the through travel lane to the top of the canal side slope nearest the road will be no less than 60 ft. for highways with design speeds of 50 mph or greater. For highways~~

~~with design speeds less than 50 mph this minimum distance shall not be less than 50 ft. for rural and urban flush shoulder highways or 40 ft. for urban curb or curb and gutter highways. When new canal or roadway alignment is required, distances greater than those above should be provided, if possible, to accommodate possible future improvements to the roadway (widening, etc.). If the minimum standards for canal hazards cannot be met, then shielding should be considered.~~

~~The RSAP is the recommended tool for evaluating the cost effectiveness of shielding roadside hazards.~~

~~D.5 Vegetation~~

~~The proper use of natural vegetation can provide valuable and economical assistance in developing aesthetic and traversable roadsides.~~

~~D.5.a Stability~~

~~The use of grass or other easily maintained, low-growing vegetation may be used on medians and roadsides. This vegetation should be carefully maintained so vehicles can safely traverse those areas.~~

~~D.5.b Drainage~~

~~Drainage swales may be protected from hazardous scouring (alteration of safe ditch contour) by the appropriate vegetation. Grass, vines, or other plants can be beneficial in stabilizing embankments to prevent erosion of material onto adjacent roadways. The appropriate use of grass or shrubbery can also aid in retarding runoff in the vicinity of the roadway, thus benefiting the overall drainage pattern.~~

~~D.5.c Environmental and Aesthetic Considerations~~

~~The use of natural grass and shrubbery for borders along roadways provides an important environmental asset. This border serves as a preserved green belt that minimizes the adverse impact (dirt, noise, etc.) of a street or highway. The use of a wide, gently flowing grassed roadside of varying width is generally an aesthetically pleasing design.~~

~~D.5.d Landscaping - Design Considerations~~

~~The Department's ***Design Standards (Index Numbers 544 - Landscape Installations, and 546 - Sight Distance at Intersections)***, contain information on landscaping that may be considered. ***Index 544*** provides landscape installation details. The Department also produces the "Florida Highway Landscaping Guide" which is an excellent landscaping information source.~~

~~***Standard Index 546*** provides information on landscaping in vicinities of conventional intersections. For roundabout landscape guidelines and related sight line requirements, refer to ***NCHRP 672 "Roundabouts: An Informational Guide***."~~

~~D.6 Drainage~~

~~Proper drainage of the pavement, shoulders, median, and roadsides is important for maintaining a safe street or highway. Techniques utilized for providing drainage should result in safe vehicle operation on or off the roadway.~~

~~D.6.a Inlets~~

~~Drainage inlets should not be placed in a bus bay, travel, or bike lane and should not be placed in a shoulder, except at the exterior edge, when drainage restrictions are severe. Drainage inlets within the median or roadside(s) shall be traversable. A small area around the inlet should be paved to improve drainage and to prevent localized erosion. Corner radii inlets should be avoided as they hinder pedestrians, create ponding, create maintenance problems, and complicate intersection design.~~

~~D.6.b Ditches~~

~~Drainage ditches perpendicular to the roadway should not be used within the median or roadsides. All drainage ditches within the median or roadsides shall meet the requirements for slopes and changes in grade given in ***Chapter 3 - Geometric Design***.~~

~~D.6.c Culverts~~

~~Where culverts are unavoidable at intersections, the entrance and exit should be flush with the adjacent ground or located beyond the clear zone. The slope and changes in grade at the structure should conform to minimum requirements for roadsides. Culvert terminations at median crossovers should be constructed in a similar fashion.~~

~~Where culverts are required perpendicular to the roadway, they should be extended to the roadsides as a minimum. Headwalls at the culvert terminations (within the clear zone) should not protrude above the ground surface in excess of 4 inches. Sloping entrances and exits generally flush with side slopes should be used wherever possible (even outside the clear zone). Proper ground contouring of the roadside approach can provide a relatively smooth surface that can be traversed with reasonable safety by an errant vehicle.~~

~~Cross drains and side drains within the clear zone should be equipped with mitered end sections.~~ ~~***FDOT Standard Index Series 200***~~ ~~provides requirements for the proper use of flared and mitered end sections.~~

~~D.7 Curbs~~

~~The basic criteria for prohibiting or permitting the use of curbs are given in Chapter 3 - Geometric Design. Curbs serve any or all of the following purposes: drainage control, roadway edge delineation, right of way reduction, aesthetics, delineation of pedestrian walkways, reduction of maintenance operations, and assistance in orderly roadside development.~~

~~Curbs should not be used along freeways or other high-speed arterials, but if a curb is needed, it should not be located closer to the traveled way than the outer edge of the shoulder. In addition, sloping end treatments should be provided.~~

~~D.8 Poles and Support Structures~~

~~The location and design of poles or support structures for signs, signals, lighting, or other purposes is an important aspect of safe roadside design. All poles and support structures should be located outside the required clear zone when practical unless their supports are of the frangible or breakaway type. Non-~~

~~breakaway poles and sign support structures may be located behind a barrier that is present for another reason. For proper offset from rigid obstacles to barriers, see section "E" of this chapter.~~

~~The function of a breakaway support is to minimize the vehicle deceleration and the probability of injury to vehicle occupants. The design of the support should also be adequate to prevent portions of the structure from penetrating the vehicle interior.~~

~~Small signs should be designed to bend over flush with the ground upon impact. Larger signs should be designed with multiple posts with slip joints at the base and a weakened section and fuse plate intended to act as a hinge at the bottom of the sign.~~

~~Utility poles and structures not related to highway operations, should be located outside the clear zone and as close as practical to the edge of right of way, without aerial encroachment, and without violating National Electric Safety Code (NESC) clearances. New utility poles not placed at the edge of the right of way, and falling within the limits of the clear zone dimensions defined in Table 3-12 should be approved through the exception process prescribed in **_Chapter 14 - Design Exceptions_**. Placement within sidewalk shall be such that a minimum unobstructed sidewalk width of 32" is provided.~~

~~In accordance with **_Section 337.403, Florida Statutes_**, existing utility poles must be relocated when **_unreasonably_** interfering with the "convenient, safe, or continuous use, or the maintenance, improvement, extension, or expansion" of public roads. Utility poles adjacent to road improvement projects, but not directly interfering with construction, should be considered for relocation, to the extent they can be relocated, to achieve the clear zone requirements of Table 3-12. Utility poles that cannot be relocated and will remain within the clear zone, should be approved through the exception process prescribed in **_Chapter 14 - Design Exceptions_**.~~

~~D.9 Intersections~~

~~All poles or other structures not absolutely essential should not be located in the vicinity of the intersection. When joint use agreements can be arranged, the various governmental agencies, transit authorities, and utilities should consider the use of joint purpose single poles as a replacement for all poles or structures serving a single purpose. Light poles, traffic signal supports and boxes, transit stop signs,~~

~~and all other street furniture should be moved back as far as is practical from the boundary of the roadsides.~~

~~Energy absorbing devices should be considered for protection of lighting and traffic signal supports located within the roadsides.~~

~~D.10 Underpasses~~

~~The full median and roadside should be carried through underpasses without interruption. Where it is not feasible to eliminate the supports, guardrail or another longitudinal barrier should be used. The barrier may be a rigid barrier incorporated into the support columns or a guardrail set out from the supports. The barrier should be extended well beyond the supports.~~

~~D.11 Bridges and Overpasses~~

~~The required lateral offset (***Chapter 3 - Geometric Design***) should be maintained on all bridges, overpasses, or other elevated roadways. The full roadway cross section, including shoulders, should be carried across without interruption. Bridge railings should be designed and constructed in compliance with the requirements for redirection barriers. Particular emphasis should be placed on the prevention of structural failure and vaulting of the railing by errant vehicles.~~

~~On all high speed roadways (design speed 50 mph or greater), the bridge railing or other barriers should be extended sufficiently (and properly terminated) to prevent vehicles from passing behind the barrier and entering the hazardous location. The transition between the bridge railing and the approach barrier should be smooth and continuous. Barrier curbs should not be placed in front of bridge railings or other barriers. Pedestrian facilities should be placed outside of the bridge railing or longitudinal barrier on all high speed roadways.~~

~~It is desirable that twin bridges for nominal width median divided highways be filled in the dividing area, carrying the median across the bridge without interruption. The gore area between diverging elevated roadways should be bridged over for a sufficient distance to allow for the placement of any energy absorbing devices. If twin bridges are used, the median layout should conform to ***Chapter 3 - Geometric Design***.~~

~~See ***Chapter 17 – Bridges and Other Structures*** for additional requirements for~~

~~bridges and bridge railings.~~

~~D.12 Mailboxes~~

~~Guidelines for the location of mailboxes, type of support and turnout construction, given in the Department's ***Design Standards, Index 532 - Mailboxes*** or ***AASHTO - "A Guide for Erecting Mailboxes on Highways"***, should be considered.~~

~~D.13 Bus Shelters~~

~~Bus shelters should be moved back as far as practical from the roadside with pedestrian access to the bus stop boarding and alighting area at the roadside.~~

E BARRIERS, END TREATMENTS AND CRASH CUSHIONS~~PROTECTIVE DEVICES~~

~~Protective devices for roadside design may be considered as highway safety features intended to reduce the severity of run-off-the-road crashes. In those situations where the minimum safety standards for median and roadside are not feasible, protective devices should be considered. Longitudinal barriers should not be used indiscriminately, for at least two reasons: they are expensive to install and maintain, and they are closer to the road than the obstacles they are shielding. They should be used when they are warranted by the reduction in crash severity.~~

~~Refer to the ***Florida DOT Plans Preparation Manual, Chapter 4 Roadside Safety*** for additional information on roadside and median barriers and crash cushions.~~

E.1 Roadside Barriers~~Redirection Devices~~

Roadside barriers are used to shield motorists from roadside hazards and in some cases are used to protect bystanders, pedestrians, cyclists and/or workers from vehicular traffic. In still other cases, roadside barriers are used to protect bridge piers from vehicle impacts. Median barriers are similar to roadside barriers but are designed for vehicles striking either side and are primarily used to separate opposing traffic on a divided highway. Median barriers also may be used on heavily traveled roadways to separate through traffic from local traffic or to separate high occupancy vehicle (HOV) and managed lanes from general-purpose lanes. Barriers are further classified as rigid, semi-rigid and flexible which are discussed in more detail below.

Barrier transition sections are used between adjoining barriers that have significantly different deflection characteristics. For example, a transition section is needed where a semi-rigid guardrail attaches to the approach end of a rigid concrete bridge rail, or when a barrier must be stiffened to shield fixed objects.

Requirements for bridge railings are provided in ***Chapter 17 – Bridges and Other Structures.***

~~Redirection devices are longitudinal barriers, such as guardrails, median barriers, and bridge railings placed parallel to the roadway to contain and redirect errant vehicles.~~

~~E.1.a Function~~

~~**The primary function of a longitudinal barrier is to redirect an errant vehicle away from hazardous roadside obstacles. The barrier should be designed to produce a minimum of adverse impacts (lateral and longitudinal) to a vehicle.**~~

~~E.1.b Warranting Conditions~~

~~**Warranting conditions for the use of longitudinal barriers are essentially those conditions in which the overall probability of injuries and fatalities would be reduced by the use of these redirection devices.** *AASHTO's Roadside Design Guide* **contains warrants related to roadside barrier selection and placement.**~~

~~E.1.c Location~~

~~**Ideally, the barrier should be located to minimize the likelihood of being struck by an errant vehicle. The barrier should be located outside the normal shoulder width. The location and orientation of the barrier should also be selected to minimize the angle of impact and**~~

~~**the resulting vehicle deceleration.**~~

~~**Barriers shall be offset from obstacles or other hazards a sufficient distance so the barrier may deflect without interference. The location of the barrier should be selected in close coordination with the design of its deflection characteristics.**~~

~~E.1.d Length~~

~~**The length of a longitudinal barrier should be sufficient to prevent a vehicle, traveling in either direction, from passing behind the barrier and striking the hazard being shielded.**~~

~~E.1.e Vehicle Containment~~

~~**Longitudinal barriers should have sufficient strength to prevent a vehicle from penetrating the barrier. Structural continuity and smoothness is also required to prevent rapid deceleration or penetration of the vehicle by any of the barrier components. The shape and height of the barrier should be adequate to deter overturning or vaulting of the vehicle. The surface in front of the barrier should be approximately perpendicular to the barrier and should be free from barrier curbs or other discontinuities.**~~

~~E.1.f Barrier Types~~

~~**Longitudinal barriers may be generally classified as rigid or flexible. The recommended barriers in the following sections are intended as general guidelines only. As new types of barriers are developed and tested successfully, they may be incorporated into roadside design. They should, however, conform with the requirements previously established.**~~

- ~~**Rigid Barrier - Rigid barriers are generally less effective in controlling lateral vehicle deceleration at locations subject to high-angle impacts. The use of this barrier is recommended for bridge railings and for use at retaining walls, rock cuts, or other rigid hazards**~~

~~**where space limitations are constrained.**~~

~~• **Flexible Barrier - Barriers which yield somewhat on impact are often more useful in limiting the rate of vehicle deceleration. Special care should be exercised to ensure they are structurally adequate and they maintain a smooth continuous surface.**~~

~~**This type of barrier can be expected to deflect 2 to 5 feet under impact. The post spacing may be increased when a stiffer rail is utilized. The weak post barrier and the cable barrier can be expected to deflect 8 to 12 feet or more and should be limited to locations with adequate clear space.**~~

~~E.1.g Transitions~~

~~**Changes in barrier types should be kept to a minimum. Transitions between two types of barriers should be smooth and continuous with no protruding components that could snag or penetrate a vehicle striking the barrier from either direction of travel. The transition from a flexible to a rigid barrier should be stiffened gradually to prevent "pocketing" of an errant vehicle.**~~

~~E.1.h Terminations~~

~~**Barrier terminations or interruptions should be kept to a minimum. The barrier termination should be designed to allow for a reasonably safe traversal by a vehicle traveling in either direction.**~~

~~Roadside guardrails should be flared away from the roadway. The use of energy absorbing devices as the termination of the longitudinal barrier is an effective and acceptable procedure for both roadsides and medians.~~

E.2 End Treatments~~Energy Absorbing Devices~~

End treatments include end anchorages, end terminals, and crash cushions. End anchorages are used to anchor a flexible or semi-rigid barrier to the ground to develop its tensile strength during an impact. End anchorages are not designed to be crashworthy for end on impacts. They are typically used on the trailing end of a roadside barrier on one-way roadways, or on the approach or trailing end of a flexible or semi-rigid barrier that is located outside the clear zone or that is shielded by another barrier system. End anchorages are discussed in more detail below.

End terminals are basically crashworthy anchorages. End terminals are used to anchor a flexible or semi-rigid barrier to the ground at the end of a barrier exposed to approaching traffic. Most end terminals are designed for vehicular impacts from only one side of the barrier, however some are designed for median applications where there is potential for impact from either side. End terminals are discussed in more detail below.

~~E.2.a Function~~

~~The primary function of an energy absorbing device or crash cushion is to reduce the severity of impacts with fixed objects. These are utilized at locations where impact with the roadside obstacle would produce a greater deceleration rate. The deceleration rate is controlled by providing a cushion which deforms and absorbs energy while bringing the vehicle gradually to a stop.~~

~~E.2.b Warranting Conditions~~

~~Crash cushions are used for the protection of occupants of an errant vehicle which might strike obstacles within the median or roadside that would produce excessive vehicle deceleration.~~

~~Other locations or situations that should be considered for crash cushions include:~~

- ~~Gore areas on elevated roadways~~
- ~~Intersections~~
- ~~Barrier terminations~~
- ~~Bridge abutments and supports~~
- ~~Retaining walls~~

- ~~Any other roadside object subject to impact by an errant vehicle~~

~~E.2.c Design Criteria~~

~~The primary design criteria are the limitation of vehicle deceleration which is a function of the vehicle speed and the total crash cushion deformation.~~

~~The crash cushion should be located as far from the roadway as is practicable to reduce the likelihood of impact. Special care should be exercised in the design to reduce the probability of a vehicle overturning or vaulting the crash cushion.~~

~~E.2.d Design Details~~

~~The development and testing of crash cushions are both recent and rapid. The rapidly expanding technology in this field requires the most recent research and experience be utilized in selecting a particular type of crash cushion.~~ ~~***AASHTOs Roadside Design Guide***~~ ~~provides guidance for the selection of sacrificial, re-useable and low maintenance crash cushion types.~~

E.3 Crash Cushions

Crash cushions, sometimes referred to as impact attenuators, are crashworthy end treatments typically attached at the approach end of median barriers, roadside barriers, bridge railings or other rigid fixed objects, such as bridge piers. Crash cushions may be used in a median, a ramp terminal gore, or other roadside application. Crash cushions are discussed in more detail below.

E.4 Performance Requirements

Roadside barriers, transitions, end terminals, and crash cushions must be crashworthy as determined by full scale crash testing in accordance with specific crash test criteria discussed in Section C. Descriptions of commonly used devices in Florida are described below. Section C also provides references where more information can be found on crashworthy devices.

E.5 Warrants

The determination as to when shielding is warranted for given hazardous roadside feature must be made on a case-by-case basis, and generally requires engineering judgment. It should be noted that the installation of roadside barriers presents a hazard in and of itself, and as such, the designer must analyze whether the installation of a barrier presents a greater risk than the feature it is intended to shield. The analysis should be completed using the ***Roadside Safety Analysis Program (RSAP)*** or in accordance with the ***AASHTO Highway Safety Manual (HSM)***.

Please see Section A for the considerations to be included when determining when to shield a roadside hazard.

The following hazards located within the clear zone are normally considered more hazardous than a roadside barrier:

E.5.a Above Ground Hazards

Above ground hazards are defined in Section B, Table 4 – 2 Lateral Offset. They include but are not limited to:

1. Bridge piers, abutments and railing ends
2. Parallel retaining walls with protrusions or other potential snagging features
3. Non-breakaway sign and lighting supports
4. Utility Poles
5. Trees greater than 4” in diameter measured 6” above ground.

E.5.b Drop-Off Hazards

Drop-off hazards are defined in Section B, Table 4-2 Lateral Offset.

E.5.c Canals and Water Bodies

Criteria for addressing canal and water body hazards is provided in Section B.2.c.

E.6 Warrants for Median Barriers

Median barriers shall be used on high speed, limited access facilities where the median width is less than the minimum values given in Chapter 3, Geometric Design, Table 3 – 16 Minimum Median Widths. For locations where median widths are equal to or greater than the minimum, median barriers are not normally considered except in special circumstances, such as a location with significant history of cross median crashes. Any determination to use a median barrier on limited access facilities must consider the need for barrier openings for median crossovers that are appropriately spaced to avoid excessive travel distances by emergency vehicles, law enforcement vehicles, and maintenance vehicles. The FDOT Design Manual may be referenced for additional criteria and guidelines for locating and designing median crossovers on limited access facilities.

On high speed divided arterials and collectors, median barriers are not normally used due to a number of factors that are very difficult, if not impractical, to address. Such factors include right-of-way constraints, property access needs, presence of at-grade intersections and driveways, adjacent commercial development, intersection sight distance and barrier end termination. However, provided these factors can be properly addressed, median barriers for these type facilities may be considered where median widths are less than minimum or where justified on the basis of significant crossover crash history.

See Section E for median barrier types and proper end treatment requirements. The ***AASHTO Roadside Design Guide*** and Department's ***Design Manual, Chapter 215 Roadside Safety*** and ***Standards Plans*** provide additional information and guidelines on the use of median barriers

~~permittedThe use of median barriers to reduce horizontal separation is permitted on facilities with substantially full control of accessconstitutesevere urban freeways andSee **Chapter 3 – Geometric Design** for criteria for median barriers~~. .

E.7 Work Zones and Temporary Barriers

Clear zone widths for work zones, as a minimum, shall be the lessor of clear zone requirements provided in Table 4 – 1 Minimum Width of Clear Zone, Table 4 – 5 Clear Zone Width Requirements for Work Zones, or existing clear zone width. Clear zone widths in work zones are measured from the edge of Traveled Way defined by the Temporary Traffic Control (TTC) Plan.

Table 4 – 5 Clear Zone Width Requirements for Work Zones

Work Zone Posted Speed (mph)	Travel Lanes & Multilane Ramps (feet)	Auxiliary Lanes & Single Lane Ramps (feet)
Curbed		
45 mph or less~~All Speeds w/Curb & Gutter~~	4' Behind Face of Curb	4' Behind Face of Curb
Flush Shoulder		
30 – 40	14	10
45 – 50	18	10
55	24	14
60 – 70	30	18

When clear zone widths cannot be met, the use of temporary barriers shall be considered. Temporary barriers in work zones can serve several functions:

- Shield edge drop-offs, excavation, roadside structures, falsework for bridges, material storage sites and/or other exposed objects.
- Provide protection for workers.
- Separate two-way traffic.
- Separate pedestrians from vehicular traffic.

The decision to use temporary barriers in a work zone should be based on engineering judgement and analysis. There are many factors, including traffic volume, traffic operating speed, offset, and duration, that affect barrier needs within work zones. The Department's ***~~Design~~******Standard Plans***, Index 102-600 Series, ***MUTCD*** and the ***AASHTO Roadside Design Guide*** provide additional information and guidance on the use of temporary barriers in work zones.

E.8 Barrier Types

Roadside barriers are classified as flexible, semi-rigid and rigid depending on their deflection characteristics when impacted. Flexible systems have the greatest deflection characteristics. Given much of the impact energy is dissipated by the deflection of the barrier and lower impact forces are imposed on the vehicle, flexible systems are generally more forgiving than rigid and semi-rigid systems. Rigid barriers, on the other hand, are assumed to exhibit no deflection under impact conditions so crash severity will likely be the highest of the three classifications.

In the following sections are ~~a~~basic descriptions of the barrier types commonly used in Florida for each these classifications. These commonly used barriers are those that are addressed in the Department's ~~FDOT Design~~ ***Standard Plans*** and ~~FDOT Plans Preparation~~ ***Design Manual***. Those documents should be referenced for additional details and discussion on the proper use of these systems.

The basis for the Department's systems and devices, as well as many other generic and proprietary guardrail systems meeting ***NCHRP Report 350*** and/or MASH criteria, can be found in the following documents:

- ***AASHTO Roadside Design Guide***
- ***Federal Highway Administration (FHWA) Countermeasures that Reduce Crash Severity***
- ~~http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware~~
- AASHTO-Associated General Contractors of America (AGC}-American Road and Transportation Builders Association (ARTBA) Joint Committee Task Force 13 report, A Guide to Standardized Highway Barrier Hardware available at http://www.tf13.org/Guides/.

E.8.a Guardrail

The most commonly used barrier on new construction projects in Florida is the w-beam guardrail system detailed in the Department's ~~***Design***~~ ***Standard Plans, Index 536-001~~400~~*** referenced as "General TL-3 Guardrail". This w-beam guardrail system, sometimes referred to as a strong post guardrail system, is a semi-rigid system, uses posts at 6'-3" spacing, 8" offset blocks, and mid-span splices with a rail height of 2'-1" to center of the panel. This system was developed based on the 31" Midwest Guardrail System (MGS) and meets MASH Test Level 3 criteria. Compatible proprietary components may be referenced by the 31" height. This system can be used as a roadside barrier or in a double face configuration as a median barrier. Deflection space requirements for this system are provided in the Department's ~~***Plans Preparation***~~ ***Design Manual, Chapter 215 Roadside Safety.***

The current 31" height system replaces the 27" height system (1'-9" to center of panel) that had been used for many years and still present on roadways throughout Florida. Section C.3 addresses requirements for upgrading existing 27" height systems.

The Department's ~~Design~~ ***Standard Plans, Index 536-001*** also provides details for a similar w-beam guardrail system referenced as "Low Speed, TL-2 Guardrail", with posts at 12'- 6" spacing which meets MASH Test Level 2 criteria. While this TL 2 system may be used on low speed roadways 45 mph or less, it preferably should be used only on roadways with design speeds 35 mph and less to account for the potential for changes in posted speed limits and/or vehicles exceeding the design speed.

To achieve a minimum level of crash performance, guardrail installations shall have a minimum length of 75 feet with design speeds greater than 45 mph.

E.8.b Concrete Barrier

The most commonly used concrete barriers in Florida are detailed in the Department's ~~***Design***~~ ***Standard Plans, Index 521-001***~~***410***~~. Details are provided for median application, shoulder application and pier protection. Additional information on these barriers is provided in the Department's ~~***Plans Preparation***~~***Design Manual, Chapter 215 Roadside Safety.***

The Department's 32" height F-Shape concrete barrier wall system that has been in use for many years meets ***NCHRP Report 350*** Test Level 4 criteria and MASH Test Level 3 criteria. The Department is replacing this 32" F-Shape system with a 38" height single slope concrete barrier system which meets MASH Test Level 4 criteria. In addition to improved crash test performance, the single slope face provides for simpler construction.

While shielding bridge piers to protect motorists from a hazard within the clear zone is often necessary, some bridge piers may need shielding for protection from damage due to design limitations (i.e. piers not designed for vehicular collision forces). Coordination with the Structural Engineer of Record is required to determine if pier protection is warranted. The Department's ***~~Design~~*** ***Standard Plans, Index 521-002~~411~~*** provides details for crashworthy Pier Protection barriers and the ***~~Plans Preparation~~*** ***Design Manual***, Chapter ***215 Roadside Safety*** provides a process for determining the appropriate level of pier protection. As with median and shoulder concrete barrier walls, the Department is replacing the F-Shape pier protection barriers that have been in use for several years with single slope face systems.

E.8.c High Tension Cable Barrier

There are a variety of crash tested flexible barrier systems using w-beam and cable, but they historically have not been in common use in Florida. In recent years several proprietary high-tension cable barrier (HTCB) systems have been developed that meet NCHRP Report 350 and MASH criteria. These systems are installed with a significantly greater tension in the cables than the generic low-tension systems that have been used in some states for many years. High tension cable barrier systems may be used for both median and roadside application. Deflection space requirements are dependent on the system, system length and post spacing, and are significantly greater than semi-rigid systems.

High tension cable barrier has shown to have several advantages over other types of flexible barrier systems. One advantage is they tend to result in less damage when impacted. Another is that certain systems have been tested for use on slopes as steep as 1:4. Still another advantage is that in many cases, the cables remain at the proper height after an impact that damages several posts. While no manufacturer claims their barrier remains functional in this condition, there is the potential that this offers a residual safety value under certain crash conditions. Posts are typically lightweight and can be installed in cast or driven sockets in the ground to facilitate removal and replacement. One disadvantage is that each vendor uses a different post design and cable arrangement, and therefore posts are not interchangeable between systems manufactured by different vendors.

The Department has used High Tension Cable Barrier (HTCB) in selected locations and continues to install these systems using the Department's ***Developmental Design Standards and Developmental Specifications (DDS)*** process. Detailed information on the usage requirements and design criteria of HTCB can be found on the ***Department's DDS*** website.

~~http://www.dot.state.fl.us/rddesign/DS/Dev.shtm.~~

It includes the following:

~~Instructions for~~ Developmental ~~Design~~Standard Plans ~~(IDDS),~~ Instructions D 540-001

Developmental Standard Plans Index D 540-001

Developmental Specification, Dev540

~~When considering the use of a Developmental Design Standards device, review the Department'sFDOT's *Developmental Design Standards Usage Process*, included on the DDS Website. (http://www.dot.state.fl.us/rddesign/DS/Dev /Developmental-Design-Standards-Usage-Process.pdf).~~

E.8.d Temporary Barrier

As stated in Section E.5.e, temporary barriers are used primarily in work zones for several purposes. The most commonly used temporary barriers in Florida are those adopted for use by FDOT. The department's~~FDOT~~ temporary barriers include:

Low Profile Barrier –~~Design~~ Standard Plans, Index 102-120~~412~~ (TL-2, NCHRP 350)

Type K Barrier –~~Design~~ Standard Plans, Index 102-110 ~~414~~ (TL-3, NCHRP 350)

Proprietary Temporary Barrier –~~Design~~ Standards Plans, Index 102-100 ~~415~~ and the ~~&~~***Approved Products List (APL)*** (TL-2 & TL-3, NCHRP 350)

Additional information on the proper use of these barriers is provided in the Department's ~~FDOT Plans Preparation~~***Design Manual*** and the Vendor drawings on the ~~FDOT~~***Approved Products List***.

Additional information on temporary barrier systems meeting ***NCHRP Report 350*** and/or MASH criteria can be found in the ***Manual for Assessing Safety Hardware*** and the ***AASHTO Roadside Design Guide***.

E.8.e Selection Guidelines

The evaluation of numerous factors is required to ensure that the appropriate barrier type is selected for a given application. Consideration should be given to the following factors when evaluating each ~~particular~~ site:

- Barrier Placement requirements (see Section E.6.f)
- Traffic characteristics (e.g. vehicle types/percentages, volume, and growth)
- Site characteristics (e.g. terrain, alignment, geometry, access facility type, access locations, design speed, etc.)
- Expected frequency of impacts
- Initial and replacement/repair costs
- Ease of maintenance
- Exposure of workers when conducting repairs/maintenance
- Aesthetics

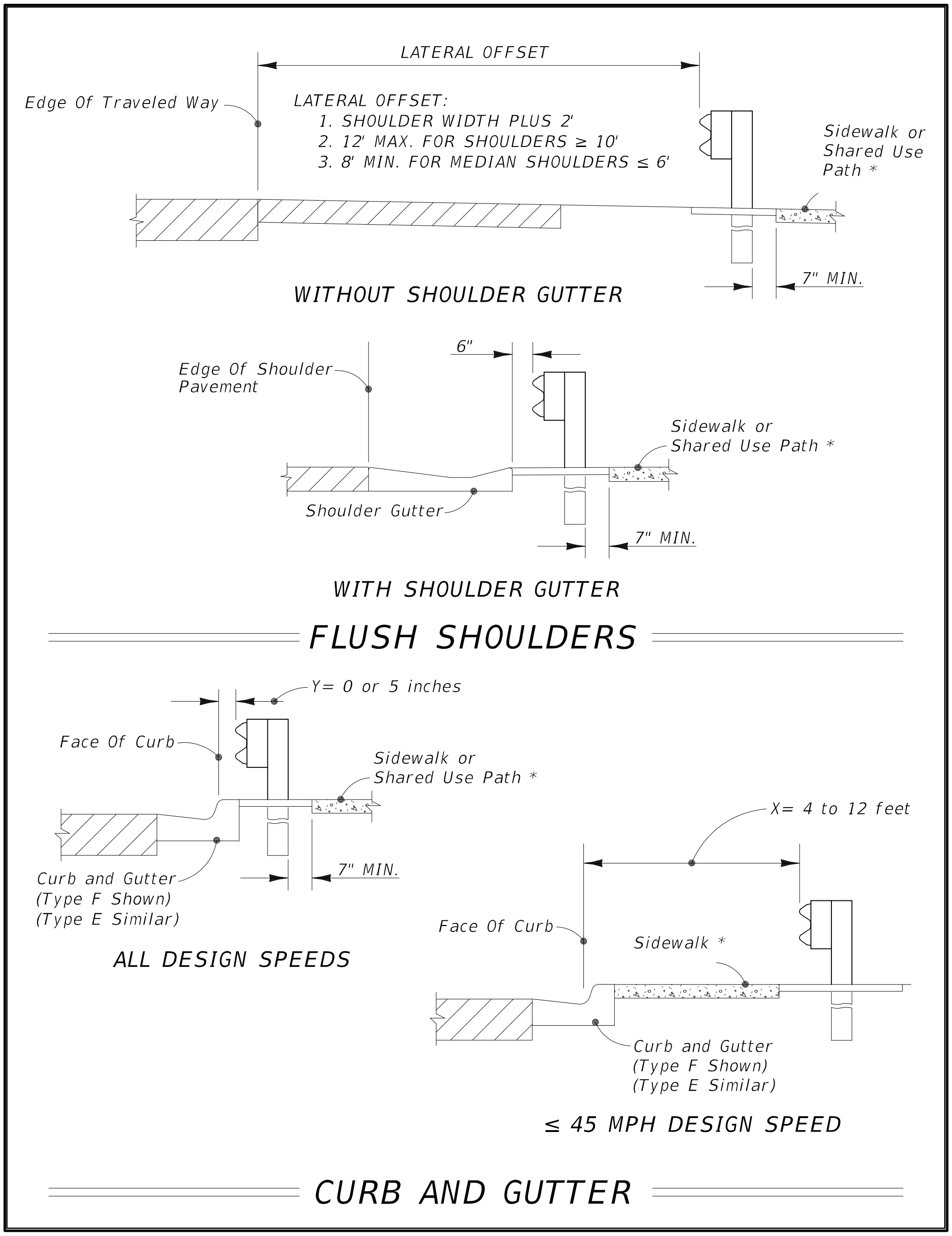
For additional information about considerations for barrier selections refer to the ***AASHTO Roadside Design Guide***. Barrier type selection decisions and warrants should be documented.

E.8.f Placement

E.8.f.1 Barrier Offsets

Roadside barriers should be offset as far from the travel lanes as practical with consideration for maintaining the proper performance of the barrier. For the ~~FDOT~~barriers described above see the Department's ***Design*** ***~~FDOT Plans Preparation~~******Manual, Chapter 215 Roadside Safety*** and~~Design~~ ***Standard Plans*** for proper barrier placement. Figure 4 – 8 Location of Guardrail provides information on the offset of guardrail on curbed and flush shoulder roadways.

Figure 4 – 8 Location of Guardrail



* When a sidewalk is present or planned. See Chapter 8 – Pedestrian Facilities and Chapter 9 – Bicycle Facilities for criteria for sidewalks and shared use paths (e.g. width of facility plus clear, graded areas adjacent to the path or sidewalk).

E.8.f.2 Deflection Space and Zone of Intrusion

In addition to travel lane lateral offset considerations, an adequate setback must be provided behind the barrier to ensure proper function. For flexible and semi-rigid barriers, the setback is based on deflection tolerances and is required to prevent the barrier from contacting aboveground objects.

For rigid barriers, the setback is required to keep the area above and behind the barrier face free of obstructions that could penetrate or damage the vehicle compartment. This requirement is based on the Zone of Intrusion (ZOI) concept as described in the ***AASHTO Roadside Design Guide.***

~~Table 4 – 6 Minimum Barrier Setback provides the setback requirements for the Department's standard barriers. Additionally, Figure 4 – 9 Setback Distances for Discontinuous Elements includes setback distances to rigid barriers for discontinuous elements.~~ These requirements do not apply to devices located within the setback distances detailed in the Department's ***Standard Plans*** ~~***Design Standards***~~(e.g. pedestrian/bicycle railing, fencing, noise walls, etc.).

~~***Plans Preparation Manual Design Standards***~~ .

E.8.f.3 Grading

The terrain effects between the traveled way and a barrier can have a significant impact on whether ~~or not~~ a barrier will perform as intended. Proper grading around a barrier will ensure that as a vehicle approaches a barrier its suspension is not dramatically affected, causing the vehicle to underride or override a barrier.

~~Department***Plans Preparation Manual Design Standards***~~.

E.8.f.4 Curbs

As with grading, the presence of curb in combination with barriers deserves special attention. A vehicle which traverses a curb prior to impact may override the barrier if it is partially airborne at the moment of impact. Conversely, the vehicle may "submarine" under the rail element of a guardrail system and snag on the support posts if it strikes the barrier too low.

~~Department's***Design Standards*** and ***Plans Preparation Manual***~~.

E.8.f.5 Flare Rate

A flared roadside barrier is when it is not parallel to the edge of the traveled way. A flared barrier may be necessary for several reasons:

- To locate the barrier terminal farther from the roadway
- To minimize a driver's reaction to an obstacle near the road by gradually introducing a parallel barrier installation
- To transition a roadside barrier to an obstacle nearer the roadway such as a bridge parapet or railing
- To reduce the total length barrier needed.
- To reduce the potential for barrier and terminal impacts and provide additional roadside space for an errant motorist to recover.

A concern with flaring a section of roadside barrier is that the greater the flare rate, the higher the angle at which the barrier can be hit. As the angle of impact increases, the crash severity increases, particularly for rigid and semi-rigid barrier systems. Another disadvantage to flaring a barrier installation is the increased likelihood that a vehicle will be redirected back into or across the roadway following an impact.

For the ~~Department's~~ barriers described above, see the Department's ***Design Manual, Chapter 215 Roadside Safety*** ~~***Design Standards***~~ ~~and~~ ~~***Chapter 4***~~ ~~of the~~ ~~***Plans Preparation Manual***~~for acceptable flare rates. Additional information on flare rates are provided in the ***AASHTO Roadside Design Guide***.

E.8.f.6 Length of Need

The length of need for a particular barrier type is calculated based on several factors including the length of the hazard, the lateral area of concern, run out length and other factors. Length of need must consider traffic from both directions.

A spreadsheet tool for calculating length of need is provided on the Department's ***Standard Plans*** ~~***Design Standards***~~web page, adjacent to ***Index 536-001*** ~~400~~ in the ***Design Tools*** column. Additional information on length of need is provided in the ***AASHTO Roadside Design Guide***.

E.8.g Barrier Transitions

Guardrail transitions are necessary whenever standard W-Beam guardrail converges with rigid barriers. The purpose of the transition is to provide a gradual stiffening of the overall approach to a rigid barrier so that vehicular pocketing, snagging, or penetration is reduced or avoided at any position along the transition. Guardrail transitions must include sound structural connections, nested panels and additional posts for increased stiffness. The Department's ***Standard Plans*** ~~***Design Standards***~~ provide details for several transitions for both permanent and rigid barriers that meet MASH criteria. Additional information on transitions is provided in the Department's ***Design Manual, Chapter 215 Roadside Safety*** and the ***AASHTO Roadside Design Guide***.

E.8.h Attachments to Barriers

Attachments to barriers such as signs, light poles, and other objects will affect crash performance and should be avoided where practical. Attachments not meeting the requirements discussed in Section E.6.f Placement, should meet crash test criteria. See the ***Department's Design Manual, Chapter 215 Roadside Safety*** ~~***Plans Preparation Manul***~~ for additional information ~~direction~~ on attachments to barriers.

E.9 End Treatments and Crash Cushions

As previously discussed, end treatments include end anchorages, end terminals, and crash cushions. Details for end treatments for each barrier type described above are detailed in the Department's ***Standard Plans*** ~~***Design Standards***~~ and the ~~Department's~~ ***Approved Products List (APL)***.

E.9.a End Treatments for Guardrail

End treatments for guardrail are categorized as follows:

1. Approach end terminals – required for guardrail ends within the clear zone of approaching traffic. The Department's guardrail approach end terminals are proprietary devices listed on the ***APL***. Approach end terminals are classified by Test Level (TL-2 for Design Speeds ≤ 45 mph or TL-3, which is acceptable for all Design Speeds) and as follows:
   a. Flared – preferred terminal for locations where sufficient space is available to offset barrier end from approaching traffic.
   b. Parallel – use only when sufficient space is not available for a flared terminal.
   c. Double Face – preferred end treatment for double faced guardrail installations.
2. Crash Cushions – See Section E.7.e.
3. Trailing End Anchorages (Type II) – required for anchoring of the trailing ends of guardrail. Trailing End Anchorages are considered non-crashworthy as an approach end treatment, and are not permitted as guardrail end treatments on the approach end within the Clear Zone, unless shielded by another run of barrier. The Department's Type II

Trailing End Anchorage, is detailed in the Standard Plans, Index 536-001 ~~Design Standards, Index 400~~.

Additional information on guardrail end treatments is provided in the Department's ***Design Manual, Chapter 215 Roadside Safety~~Plans Preparation Manual~~***.

E.9.b End Treatments for Rigid Barrier

Rigid Barrier ends must be terminated by either transitioning into another barrier system (e.g. guardrail), or by shielding with a Crash Cushion. Details are provided in the Department's ***Standard Plans, Index 521-001*** ***~~Design Standards~~***. Treatment of the trailing end of rigid barriers is not required unless additional hazards exist beyond the rigid barrier or the barrier is within the clear zone of opposing traffic.

E.9.c End Treatments for High Tension Cable Barrier (HTCB)

End treatments for high tension cable barrier are vendor specific. For additional information regarding the end treatment of HTCB, refer to the Department's developmental design standards discussed above.

E.9.d End Treatments for Temporary Barrier

Details for end treatments for the Department's Temporary Barrier are provided in the Department's ***Standard Plans*** ***~~Design Standards~~*** and include:

1. Connecting to an existing barrier. Smooth, structural connections are required. Information on connections can be found in the Department's ***Standard Plans, Indexes 521-001*** and ***102-110*** ***~~Design Standards Indexes 410 and 414~~***and ***APL).***
2. Shield end with a crash cushion as detailed in the ***Standard Plans, Index 102 Series*** ***~~Design Standards~~*** or ***APL*** for the specific type of Temporary Barrier (i.e. portable concrete barrier, steel, or water filled).
3. Attaching or Transitioning to a crashworthy end treatment as described above.
4. Flaring outside of the Work Zone Clear Zone.

E.9.e Crash Cushions

Crash cushions are classified based on Test Level and Design Speed which is shown for each system on each vendor's respective drawings posted on the Department's ***APL***.

The design of a crash cushion system must not create a hazard to opposing traffic. The APL drawings provide details for transitions for optional barrier types with and without bi-directional traffic.

An impacting vehicle should strike the systems at normal height, with the vehicle's suspension system neither collapsed nor extended. Therefore, the terrain surrounding crash cushions must be relatively flat (i.e. 1:10 or flatter) in advance of and along the entire design length of the system. Curbs should not be located within the approach area of a crash cushion.

The Department's ***Design Manual, Chapter 215 Roadside Safety*** ~~***Plans Preparation Manual***~~ provides additional information on permanent and temporary crash cushions.

F BRIDGE RAILS

See ***Chapter 17 - Bridges and Other Structures*** for requirements for bridge rails. The Department's ***Design Manual, Chapter 215 Roadside Safety*** ~~***Plans Preparation Manual***~~ may be referenced for additional information and typical applications.

G~~F~~ REFERENCES ~~FOR INFORMATIONAL PURPOSES~~

The following is a list of publications that may be referenced for further guidance:

- AASHTO Roadside Design Guide
  https://bookstore.transportation.org/
- Task Force 13 Guide to Standardized Roadside Safety Hardware
  http://www.tf13.org/Guides/
- FHWA Web Site
  http://safety.fhwa.dot.gov/roadway_dept/
- ~~NCHRP Report 672 – Roundabouts: An Informational Guide, Second Edition~~
  ~~http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_672.pdf~~
- ~~Section 401, Florida Statutes~~
  ~~http://www.leg.state.fl.us/Statutes/index.cfm?App_mode=Display_Statute&Search_String=&URL=0300-0399/0337/Sections/0337.401.html~~
- FDOT Design Manual~~Plans Preparation Manual~~
  http://www.fdot.gov/roadway/FDM/
- ~~http://www.fdot.gov/roadway/PPMManual/PPM.shtm~~FDOT Standard Plans for Road and Bridge Construction IStandard Plans) ~~Design Standards~~
  http://www.fdot.gov/design/standardplans/
  ~~http://www.fdot.gov/roadway/DesignStandards/Standards.shtm~~
- FDOT Structures Design Guidelines
  http://www.fdot.gov/structures/StructuresManual/CurrentRelease/StructuresManual.shtm
- FDOT Drainage Manual, January 2018
  http://www.fdot.gov/roadway/Drainage/ManualsandHandbooks.shtm
- Florida Strategic Highway Safety Plan 2016
  http://www.fdot.gov/safety/SHSP2016/SHSP-2012.shtm
  ~~http://www.fdot.gov/safety/SHSP2012/SHSP-2012.shtm~~

CHAPTER 6

LIGHTING

TABLES

FIGURES

CHAPTER 6

LIGHTING

A INTRODUCTION

The major reason for lighting streets and highways is to improve safety for vehicular and pedestrian traffic. Improvements in sight distance and reduction of confusion and distraction for night time driving can reduce the hazard potential on streets and highways. There is evidence indicating that highway lighting will produce an increase in highway capacity as well as improve the economic, safety, and aesthetic characteristics of highways.

Experience and technical improvements have resulted in improved design of lighting for streets and highways. Photometric data provide a basis for calculation of the illumination at any point for various combinations of selected luminaire types, heights, and locations. Lighting engineers can develop lighting systems that will comply with the requirements for level and uniformity of illumination; however, some uncertainties preclude the adoption of rigid design standards. Among these uncertainties is the lack of understanding ~~in the area~~ of driver response and behavior under various lighting conditions. The design of lighting for new streets and highways, as well as improvements on existing facilities, should be accompanied by careful consideration of the variables involved in driver behavior and problems peculiar to particular locations.

Rights of way with pedestrian sidewalks and/or bikeways adjacent to the roadway should first address lighting requirements for the roadway to assure it is continuously illuminated. Additional lighting for a sidewalk or shared use path maybe necessary if it is substantially set back from the roadway, at the discretion of the responsible/maintaining agency. Pedestrian sidewalks and/or bikeways should not be illuminated in lieu of lighting the adjacent roadway ~~in order~~ to avoid glare or potential lighting distractions to drivers.

See ***Chapter 17 – Bridges and Other Structures, Section C.6*** for structural requirements for lighting.

B OBJECTIVES

The objective for providing lighting is to improve the safety of roadways, sidewalks, and shared use paths and visibility of signs for road users (drivers, pedestrians, and bicyclists). The achievement of this objective will be aided by meeting these specific goals:

- Provide an improved view of the general highway geometry and the adjacent environment.
- Increase the sight distance to improve response to hazards and decision points.
- Eliminate "blind" spots unique to travel at night or in low light conditions.
- Provide a clearer view of the general situation during police, emergency, maintenance, and construction operations.
- Provide assistance in roadway, sidewalk or path delineation, particularly in the presence of confusing background lighting (i.e., surrounding street and other area lighting confuses the driver on an unlighted street or highway).
- Minimize glare that is discomforting or disabling.
- Reduce abrupt changes in light intensity.
- Avoid the introduction of roadside hazards resulting from improper placement of light poles, pull boxes, etc. (as covered under ***Chapter 3 – Geometric Design*** and ***Chapter 4 – Roadside Design***).

C WARRANTING CONDITIONS

Although precise warrants for the provision of roadway lighting are difficult to determine, criteria for lighting is established and should be followed for new and reconstruction projects and for improvement of existing facilities. The following locations should be considered as a basis for warranting roadway lighting:

C.1 Criteria Based Upon Crash History

- ~~Locations where pedestrians assemble to board or depart from transit services.~~
- Locations that, by a crash investigation program, have been shown to be hazardous due to inadequate lighting.
- Locations where the night/day ratio of serious crashes is higher than the average of similar locations.
- Specific locations that have a significant number of night time crashes and where a large percentage of these night time crashes result in injuries or fatalities.

C.2 Criteria Based Upon Analysis and Investigation

- Locations requiring a rapid sequence of decisions by the road user.
- Locations where night sight distance problems exist, with ~~particular~~ consideration to headlight limitations (i.e., where vertical and horizontal curvature adversely affect illumination by headlamps).
- Locations having discomforting or disabling glare.
- Locations where background lighting exists, particularly if this could be distracting or confusing to the road user.
- Locations where improved delineation of the highway alignment is needed.

C.3 General Criteria

- Roundabouts and signalized intersections.
- Urban ~~collector~~ streets, particularly with high speed, high volumes, or frequent turning movements.
- Urban streets of any category experiencing high night time volumes or speeds or that have frequent signalization or turning movements.
- Areas frequently congested with vehicular and/or pedestrian traffic.
- Pedestrian and bicyclist crossings (intersections or mid-block locations)~~,~~
- Transit stops and hubs, passenger rail stations. ~~Locations where pedestrians assemble to board or depart from transit services.~~
- ~~and a~~Areas such as entertainment districts, sporting arenas, shopping centers, beach access points, parks, and other locations that generate higher volumes of pedestrian activity.
- Schools, places of assembly, ~~transit stops~~, or other pedestrian or bicyclist generators.
- High density land use areas.
- Central business districts.
- Junctions of major highways in rural areas.
- Rest areas/picnic shelters/trail heads/recreational facilities.

D TYPES OF LUMINAIRES~~ILLUMINATION~~

Examples of common types of lighting are identified and discussed below. Other types of lighting may be desired and currently in use for specific applications.

- Light Emitting Diode (LED) – is the preferred ~~most commonly used~~ light source for street lighting. Light produced by LED lamps have a CCT of 4000°K to 6000°K which is a white to bluish color. The average rated life for LED can vary from 50,000 to 100,000 hours. To provide sufficient lumen levels for roadway applications, most LED fixtures have an initial luminous efficiency of around 75 lumens per watt.

- High Pressure Sodium (HPS) Lamps – ~~is the most commonly used light source for street lighting.~~ Light produced by HPS lamps has a correlated color temperature (CCT) around 2100°K which is a warm yellow color. The average rated life for an HPS lamp is from 24,000 to 30,000 hours. HPS lamps have a very high initial luminous ~~efficacy~~ efficiency of over 100 lumens per watt.

- Metal Halide (MH) Lamps – is ~~commonly~~ used for overhead lighting of commercial parking lots, sports facilities, retail stores and street lighting. Light produced by MH lamps has a CCT of 3800°K to 4000°K which is a white color. The average rated life of a MH lamp can vary from 9,000 to 20,000 hours. MH lamps have a high initial luminous ~~efficacy~~ efficiency of around 75 - 100 lumens per watt.

- ~~Light Emitting Diode (LED) – although LED was developed in the early 1960s, it has only recently entered the roadway lighting market. Light produced by LED lamps have a CCT of 4000°K to 6000°K which is a white to bluish color. The average rated life for LED can vary from 50,000 to 100,000 hours. The wide variation in rated life for LED's is due to the limited lumen output of a single LED. To provide sufficient lumens for roadway lighting requires that fixtures have a large number of LED's. To maximize the lumen output of each LED, fixture manufactures may use a variety of techniques to increase the lumen output such as increasing the CCT and increasing the drive current. Increasing the CCT from 3500°K to 4500°K results in an 8% increase in lumen level, however above 4500°K the rate of increase doubles. Increasing the CCT also improves the efficacy of LED's. LED's are most efficient at drive currents of 350mA or 525mA, however they can be driven as high as 2100mA. A 25% increase in lumen level can be achieved by increasing the drive current from 525mA to 700mA. The increase in lumen level drops slightly to 21% for each 175mA increase from 700mA to 1400mA. Above 1400mA, the increase in lumen level drops to 6% for each 175mA. Increasing the drive current to LED's has two serious consequences, it~~

~~substantially reduces the average rated life and the efficiency of the LED. To provide sufficient lumen levels for roadway applications, most LED fixtures have an initial luminous efficacy of around 75 lumens per watt.~~

E LIGHTING DESIGN TECHNIQUES~~LEVEL OF ILLUMINATION~~

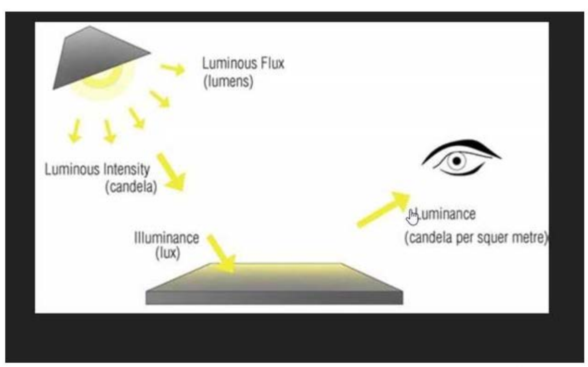
The accepted methods for achieving a given lighting condition are known as either level of illuminance or level of luminance. Both methods of calculation are dependent upon light being reflected toward the observer's eye. The luminance method is the preferred design method used for straight roadways and streets, based upon the appropriate choice of surface type. Horizontal illuminance is used for intersections and interchanges and includes a variable for surface type. Horizontal and vertical illuminance is the preferred method for pedestrian areas.

~~It is recommended that the level of illumination for streets and highways not be less than:~~

- ~~Levels consistent with need and resources.~~
- ~~Guidelines found in Table 6 – 1 Level of Illumination for Streets and Highways on the following page.~~
- ~~Lighting of mid-block pedestrian crossings at 2.0 foot candles of average maintained *vertical* illumination should be provided when night time pedestrian activity is expected.~~

Figure 6 – 1 Illuminance and Luminance illustrates how illuminance and luminance are measured. Illuminance is the measure of the amount of light flux falling on a surface and is measured in foot candles. Luminance is a measure of the amount of light flux leaving a surface and is measured in candelas per meter squared.

Figure 6 – 1 Illuminance and Luminance



E.1 Illuminance

~~When adding supplemental lighting for pedestrian activity, ensure lighting is compatible with any existing lighting in the corridor and minimizes glare.~~ The i~~I~~lluminance method ~~in roadway lighting~~ determines the amount of light incident on the roadway surface or on vertical surfaces from the roadway lighting system~~is a measure of the light at the pavement surface~~. Because the amount of light seen by the driver is the portion that reflects from the pavement towards the driver, and because different pavements exhibit varied reflectance characteristics, different illuminance levels are needed for each type of standard roadway surface. Illuminance is easily calculated and measurable and is not observer or pavement dependent.

E.2 Luminance

The l~~L~~uminance method ~~in roadway lighting~~ determines how “bright” the road is by determining the amount of light reflected from the pavement in the direction of the driver. It uses the reflective characteristics (R-classification) noted in Table 6 – 1 Road Surface Classifications for the standard roadway surface types and a specific observer position.

The R-classification system is a measure of the lightness (white to black) and specularity (shininess) of roadway surfaces. ~~is a measure of the reflected light from the pavement surface that is visible to the motorist’s eye.~~ A system of pavement reflectance values divides the pavement characteristics into four categories: R1, R2, R3 and R4. These categories are based upon the ***American National Standard Practice for Roadway Lighting*** and have been adopted by ***AASHTO*** in their ***Roadway Lighting Design Guide***. ~~They are described in Table 6 – 2 Road Surface Classifications.~~

Table 6 – 1~~2~~ Road Surface Classifications

Class	Q_0*	Description	Mode of Reflectance
R1	0.10	Portland cement concrete road surface. Asphalt road surface with a minimum of 12% of the aggregates composed of artificial brightener or aggregates.	Mostly diffuse
R2	0.07	Asphalt road surface with an aggregate composed of minimum 60% gravel (size greater than 0.4 in.). Asphalt road surface with 10 to 15% artificial brightener in aggregate mix. (Not normally used in North America).	Mixed (diffuse and specular)
R3	0.07	Asphalt road surface (regular and carpet seal) with dark aggregates (e.g., trap rock, blast furnace slag); rough texture after some months of use typical highways).	Slightly specular.
R4	0.08	Asphalt road surface with very smooth texture.	Mostly specular.

* Q_0 = representative mean luminance coefficient.

E.3 Lighting Design Levels

The level of illumination for streets and highways should not be less than those shown in Table 6 – 2 Illuminance and Luminance Design Values. ~~See Table 6 – 1 for ranges of illumination.~~ When adding supplemental lighting for pedestrian activity, ensure lighting is compatible with any existing lighting in the corridor and minimizes glare.

These levels are for the purpose of highway safety and do not apply to lighting levels required for crime reduction. Further information may be found in the ***AASHTO Roadway Lighting Design Guide (2005).***

Table 6 – 2~~1~~ Illuminance and Luminance Design Values ~~Level of Illumination for Streets and Highways~~

Roadway and Walkway Classification	Off-Roadway Light Sources	Illuminance Method					Luminance Method			Additional Values (both Methods)
		Average Maintained Illuminance (Horizontal)				Illuminance Uniformity Ratio	Average Maintained Luminance			Veiling Luminance Ratio
		R1	R2	R3	R4		Lavg	Uniformity		
	General Land Use	(foot -candles) (min)	(foot-candles) (min)	(foot-candles) (min)	(foot-candles) (min)	avg/min (max) (6)	cd/m2 (min)	Lavg/Lmin (max)	Lmax/Lmin (max)	Lv(max)/Lavg (max)[3]
Principal Arterials (partial or no control of access)	Commercial	1.1	1.6	1.6	1.4	3:1	1.2	3:1	5:1	0.3:1
	Intermediate	0.8	1.2	1.2	1.0	3:1	0.9	3:1	5:1	0.3:1
	Residential	0.6	0.8	0.8	0.8	3:1	0.6	3.5:1	6:1	0.3:1
Minor Arterials	Commercial	0.9	1.4	1.4	1.0	4:1	1.2	3:1	5:1	0.3:1
	Intermediate	0.8	1.0	1.0	0.9	4:1	0.9	3:1	5:1	0.3:1
	Residential	0.5	0.7	0.7	0.7	4:1	0.6	3.5:1	6:1	0.3:1
Collectors	Commercial	0.8	1.1	1.1	0.9	4:1	0.8	3:1	5:1	0.4:1
	Intermediate	0.6	0.8	0.8	0.8	4:1	0.6	3.5:1	6:1	0.4:1
	Residential	0.4	0.6	0.6	0.5	4:1	0.4	4:1	8:1	0.4:1
Local	Commercial	0.6	0.8	0.8	0.8	6:1	0.6	6:1	10:1	0.4:1
	Intermediate	0.5	0.7	0.7	0.6	6:1	0.5	6:1	10:1	0.4:1
	Residential	0.3	0.4	0.4	0.4	6;1	0.3	6:1	10:1	0.4:1
Alleys	Commercial	0.4	0.6	0.6	0.5	6:1	0.4	6:1	10:1	0.4:1
	Intermediate	0.3	0.4	0.4	0.4	6:1	0.3	6:1	10:1	0.4:1
	Residential	0.2	0.3	0.3	0.3	6:1	0.2	6:1	10:1	0.4:1

~~Continued on~~Continued next page

Table~~ABLE~~ 6 – 2~~1~~
Illuminance and Luminance Design Values ~~Level of Illumination for Streets and Highways~~
(Continued)

Sidewalks	Commercial	0.9	1.3	1.3	1.2	3:1	Use illuminance requirements
	Intermediate	0.6	0.8	0.8	0.8	4:1	
	Residential	0.3	0.4	0.4	0.4	6:1	
Pedestrian Ways and Bicycle Ways(2)	All	1.4	2.0	2.0	1.8	3.1	
Notes	1. Meet either the Illuminance design method requirements or the Luminance design method requirements and meet veiling luminance requirements for both illuminance and Luminance design methods. 2. Assumes a separate facility. For Pedestrian Ways and Bicycle Ways adjacent to roadway, use roadway design values. Use R3 requirements for walkway/bikeway surface materials other than the pavement types shown. 3. Lv (max) refers to the maximum point along the pavement, not the maximum in lamp life. The Maintenance factor applies to both the Lv term and the Lavg term. 4. There may be situations when a higher level of illuminance is justified. The higher values for freeways may be justified when deemed advantageous by the agency to mitigate off-roadway sources. 5. Physical roadway conditions may require adjustment of spacing determined from the base levels of illuminance indicated above. 6. Higher uniformity ratios are acceptable for elevated ramps near high-mast poles. 7. See AASHTO publication entitled, "A Policy on Geometric Design of Highways and Streets" for roadway and walkway classifications. 8. R1, R2, R3 and R4 are Road Surface Classifications, defined in the AASHTO Roadway Lighting Design Guide and further described in Table 6.2.						

~~A system of pavement reflectance values divides the pavement characteristics into four categories: R1, R2, R3 and R4. These categories are based upon the ***American National Standard Practice for Roadway Lighting*** and have been adopted by ***AASHTO*** in their ***Roadway Lighting Design Guide***. They are described in Table 6 – 2 Road Surface Classifications.~~

~~Table 6 – 2 Road Surface Classifications~~

~~Class~~	~~Q_0*~~	~~Description~~	~~Mode of Reflectance~~
~~R1~~	~~0.10~~	~~Portland cement concrete road surface. Asphalt road surface with a minimum of 12% of the aggregates composed of artificial brightener or aggregates.~~	~~Mostly diffuse~~
~~R2~~	~~0.07~~	~~Asphalt road surface with an aggregate composed of minimum 60% gravel (size greater than 0.4 in.).~~ ~~Asphalt road surface with 10 to 15% artificial brightener in aggregate mix. (Not normally used in North America).~~	~~Mixed (diffuse and specular)~~
~~R3~~	~~0.07~~	~~Asphalt road surface (regular and carpet seal) with dark aggregates (e.g., trap rock, blast furnace slag); rough texture after some months of use typical highways).~~	~~Slightly specular.~~
~~R4~~	~~0.08~~	~~Asphalt road surface with very smooth texture.~~	~~Mostly specular.~~

~~* Q_0 = representative mean luminance coefficient.~~

F UNIFORMITY OF ILLUMINATION

~~In order t~~To avoid vision problems due to varying illumination, it is important to maintain illumination uniformity over the roadway. It is recommended the ratio of the average to the minimum initial illumination on the roadway be between 3:1 to 4:1.

A maximum to minimum uniformity ratio of 10:1 should not be exceeded. It is important to allow time for the driver's eye to adjust to lower light levels. The first light poles should be located on the side of the incoming traffic approaching the illuminated area. The eye can adjust to increased or increasing light level more quickly. In transition from a lighted to an unlighted portion of the highways, the level should be gradually reduced from the level maintained on the lighted section. This may be accomplished by having the last light pole occur on the opposite roadway. The roadway section following lighting termination should be free of hazards or decision points. Lighting should not be terminated before changes in background lighting or roadway geometry, or at the location of traffic control devices.

It is also important to ensure color consistency when lighting a highway/pedestrian corridor. Mixing of different types of lighting may reduce the lighting uniformity. As we transition to LED, it is acceptable to have mixed lighting segments along the same corridor.

The use of spot lighting at unlit intersections with a history of nighttime crashes is an option.

Close coordination between the Engineer of Record and the responsible local governmental agency is essential.

G UNDERPASSES AND OVERPASSES

One of the criteria to be followed to determine requirements for underpass lighting is the relative level between illumination on the roadway inside and outside of the underpass. The height, width, and length of the underpass determines the amount of light penetration from the exterior.

The need for l~~L~~ighting of independent sidewalks or shared use paths should be evaluated on a project specific basis. Considerations include the likelihood of night time use, the role of the facility in the community's bicycle and pedestrian network, and whether alternatives are available for night time travel.

When lighting an underpass, use a wall-mounted luminaire that is attached to a pier, pier cap, or the wall copings underneath the bridge.

G.1 Daytime Lighting

A gradual decrease in the illumination level from day time level on the roadway, sidewalk or path to the underpass should be provided. Consider daytime lighting for vehicles in underpasses greater than 80 feet in length. ~~Supplemental day time lighting is normally not needed in underpasses less than 100 feet in length.~~

Supplemental lighting of sidewalks or shared use paths in roadway underpasses less than 80 ~~100~~ feet in length should be considered. Sidewalks and shared use paths on independent alignments with little natural light, especially if the exit is not visible upon entry. should be illuminated.

G.2 Night Lighting

The night time illumination level in the underpass of the roadway should be maintained near the night time level of the approach roadway. Lighting of sidewalks or shared use paths adjacent to roadways in underpasses should be considered. Sidewalks and shared use paths on independent alignments open to travel during darkness should be illuminated. .~~, sidewalk or path.~~ Due to relatively low luminaire mounting heights in underpasses, care should be exercised to avoid glare.

H DECORATIVE ROADWAY~~AESTHETIC~~ LIGHTING

Decorative or architectural roadway ~~Aesthetic~~ lighting is acceptable provided it meets the minimum design criteria and the objectives contained in this Manual. ~~chapter.~~ Examples include architectural lighting posts, cross arms, wall brackets, bollards, and light fixtures.

~~H~~I ADAPTIVE LIGHTING

Some locations such as coastal roadways where sea turtles may be affected, may require lower lighting levels and different colors than what might normally be provided. FHWA's publication ***The Guidelines for the Implementation of Reduced Lighting on Roadways*** describes a process by which an agency or a lighting designer can select the required lighting level for a road or street and implement adaptive lighting for a lighting installation or lighting retrofit. This document supplements existing lighting guidelines.

~~I~~J OVERHEAD SIGN LIGHTING

If the visibility of the sign due to roadway geometry or retro reflectivity of the sign sheeting is inadequate, overhead sign lighting should be provided. It is recommended that the level of illumination for overhead signs not be less than guidelines found in Table 6 – 3 Illuminance and Luminance Levels for Sign Lighting. See ***Chapter 18 – Signing and Marking*** for signage retroreflectivity requirements.

Table 6 – 3
Illuminance and Luminance Levels for Sign Lighting*

Ambient Luminance	Sign Illuminance		Sign Luminance**	
	Footcandles	Lux	Candelas per Square Meter	Candelas per Square Foot
Low	10 - 20	100 - 200	22 - 44	2.2 – 4.4
Medium	20 - 40	200 - 400	44 - 89	4.4 – 8.9
High	40 - 80	400 - 800	89 - 78	89 – 178

Source: AASHTO Roadway Lighting Design Guide (October, 2005), Table 10 – 1 Illuminance and Luminance Levels for Sign Lighting~~*Adapted from The IESNA Lighting Handbook, Reference & Application, 9th Edition, Illuminating Engineering Society of North America~~.

**Based upon a maintained reflectance of 70 percent for white sign letters.

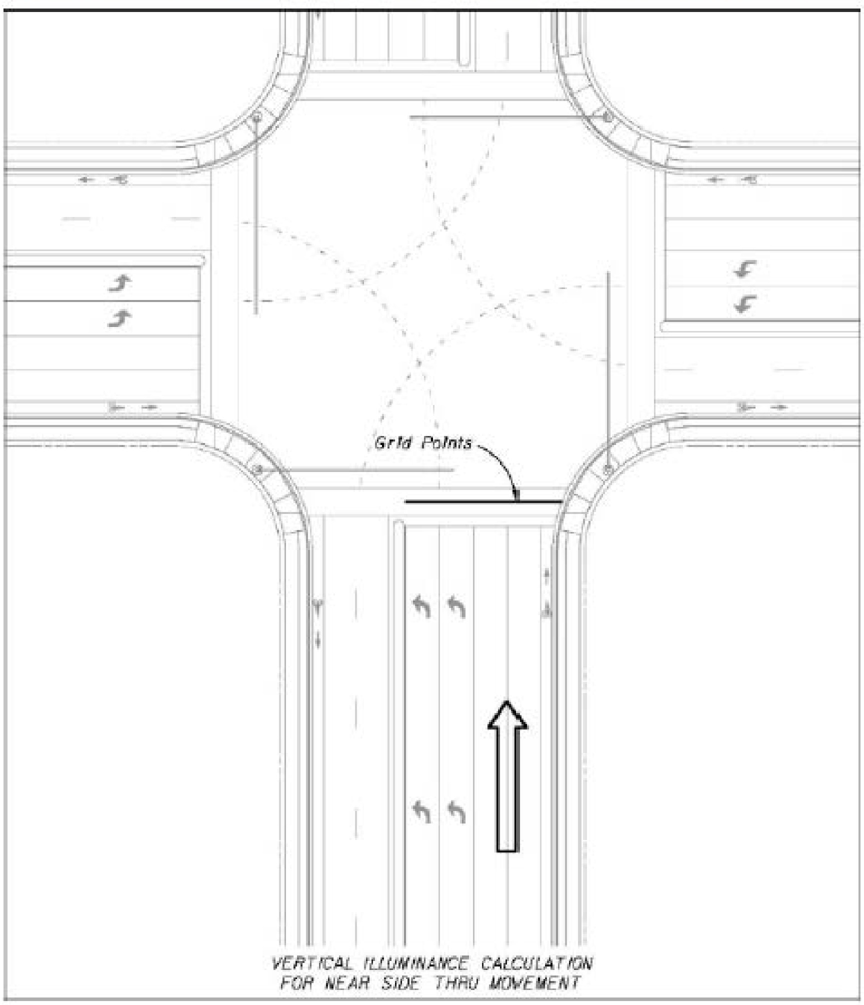
K ROUNDABOUTS

Roundabouts should be supplemented with ~~Use conventional~~ roadway lighting~~. criteria for where pedestrian traffic is not anticipated.~~. Where pedestrians are expected, provide additional lighting of 2.0-foot candles of maintained vertical illumination, measured at 5 feet from the road surface.~~lighting of the entire pedestrian crossing at 2.0-foot candles of averagemaintained vertical illumination is required~~ Calculate the vertical illuminance for the crosswalk on each near side approach entering and exiting the roundabout~~and for each right turn movement~~.

L MIDBLOCK CROSSWALKS

At midblock pedestrian crossings, provide 2.0-foot candles of maintained vertical illumination, measured at 5 feet from the road surface. ~~of average maintained vertical illumination should be provided. when pedestrian activity is expected.~~ Calculate the vertical illuminance for the crosswalk on each near side approach.

Figure 231.3.1 Vertical Illuminance Calculation for Near Side Movement



~~J~~M MAINTENANCE

A program of regular preventive maintenance should be established to ensure levels of illumination do not go below required values. The program should be coordinated with lighting design to determine the maintenance period. Factors for consideration include a decrease in lamp output, luminaire components becoming dirty, and the physical deterioration of the reflector or refractor. The maintenance of roadway lighting should be incorporated in the overall maintenance program specified in ***Chapter 10 – Maintenance and Resurfacing***.

~~K~~N LIGHT POLES

Light poles should not be placed in the sidewalk when adequate right of way is available beyond the sidewalk. Placement of lighting structures and achieved illumination may be limited by existing conditions such as driveways, overhead and underground utilities, drainage structures, and availability of right of way.

Light poles should not be placed so as to provide a hazard to ~~out of control~~ errant vehicles. Non-frangible light poles should be placed outside of the clear zone. They should be as far removed from the travel lane as possible or behind adequate guardrail or other barriers. Light poles should be placed on the inside of the curves when feasible. Foundations or light poles and rigid auxiliary lighting components that are not behind suitable barriers should be constructed flush with or below the ground level.

The use of high mast lighting should be considered, particularly for lighting interchanges and other large plaza areas. This use tends to produce a more uniform illumination level, reduces glare, and allows placement of the light poles farther from the roadway. Additional emphasis lighting should be considered to illuminate specific and desired pedestrian crossings.

The placement of light poles should not interfere with the driver's sight distance or visibility of signs, signals, or other traffic control devices. In addition, the ***National Electrical Code (NEC)*** requires a working area for safety purposes around the poles. Further criteria regarding the placement of roadside structures, including light poles, is specified in ***Chapter 4 – Roadside Design***.

~~L~~O REFERENCES

The publications referenced in this chapter can be obtained at the following web sites.

- Roadway Lighting, ANSI/RP-8-14
  http://www.ies.org/store/product/roadway-lighting-ansiies-rp814-1350.cfm

- Design Guide for Residential Street Lighting (2015), Illuminating Engineering Society
  https://www.ies.org/store/design-guides/design-guide-for-residential-street-lighting/

- AASHTO - Roadway Lighting Design Guide (October 2005)
  https://bookstore.transportation.org

- Guidelines for the Implementation of Reduced Lighting on Roadways
  PUBLICATION NO. FHWA-HRT-14-050 JUNE 2014
  http://www.fhwa.dot.gov/publications/research/safety/14050/14050.pdf

- The Lighting Handbook, 10th Edition, Illuminating Engineering Society (IESA)
  https://www.ies.org/store/lighting-handbooks/lighting-handbook-10th-edition/

- National Electric Code
  https://www.nfpa.org/NEC/About-the-NEC/Free-online-access-to-the-NEC-and-other-electrical-standards

CHAPTER 7

RAIL-HIGHWAY CROSSINGS

TABLES

FIGURES

CHAPTER 7

RAIL-HIGHWAY CROSSINGS

A INTRODUCTION

The basic design for grade crossings should be similar to that given for highway intersections in ***Chapter 3 – Geometric Design***. Rail-highway grade crossings should be limited in number and should, where feasible, be accomplished by grade separations. Where at-grade crossings are necessary, adequate traffic control devices and proper crossing design are required to limit the probability of crashes.

B OBJECTIVE AND PRIORITIES

The primary objective in the design, construction, maintenance, and reconstruction of rail-highway crossings is to provide safety for both rail and roadway vehicles in a feasible and efficient manner. The achievement of this objective may be realized by utilizing the following techniques in the listed sequence of priority.

B.1 Conflict Elimination

The elimination of at grade rail-highway conflicts is the most desirable procedure for promoting safe and efficient traffic operations. This may be accomplished by the closing of a crossing or by utilizing a grade separation structure.

B.2 Hazard Reduction

The design of new at-grade crossings should consider the objective of hazard reduction. In addition, an effective program of reconstruction should be directed towards reducing crash potential at existing crossings.

The regulation of intersections between railroads and all public streets and highways in Florida is vested in the ***Florida Administrative Code, (Rule Chapter 14-57: Railroad Safety and Clearance Standards, and Public Railroad-Highway Grade Crossings.*** This rule contains minimum requirements for all new grade crossings.

The Department's rail office has other documents available that contain additional guidance for the design, reconstruction, and upgrading of existing rail-highway grade crossings, and may be contacted for further information.

C RAIL-HIGHWAY GRADE CROSSING NEAR OR WITHIN PROJECT LIMITS

Federal-aid projects must be reviewed to determine if a rail-highway grade crossing is within the limits of or near the terminus of the project. If such rail-highway grade crossing exists, the project must be upgraded to meet the requirements of the ***Manual on Uniform Traffic Control Devices (2009 Edition with Revision Numbers 1 and 2, May 2012) (MUTCD)*** in accordance with ***Title 23, United States Code (U.S.C.), Chapter 1, Section 109(e)*** *and* ***23 C.F.R. 646.214(b).***

These requirements are located in ***Chapter 8*** of the ***MUTCD***. "Near the terminus" is defined as being either of the following:

- If the project begins or ends between the crossing and the MUTCD-mandated advanced placement distance for the advanced (railroad) warning sign. See ***MUTCD, Table 2C-4 (Condition B, Column "0" mph)*** for this distance.
- An intersection traffic signal within the project is linked to the crossing's flashing light signal and gate.

D DESIGN OF RAIL-HIGHWAY CROSSINGS

The primary requirement for the geometric design of a grade crossing is that it provides adequate sight distance for the motorist to make an appropriate decision as to stop or proceed at the crossing.

D.1 Sight Distance

The minimum sight distance requirements for streets and highways at rail-highway grade crossings are similar to those required for highway intersections (***Chapter 3 – Geometric Design***).

D.1.a Stopping Sight Distance

The approach roadways at all rail-highway grade crossings should consider stopping sight distance no less than the values given in ***Chapter 3,*** Table 3 – 3 Minimum Stopping Sight Distances for the approach to stop signs. This distance shall be measured to a stopping point prior to gates or stop bars at the crossing, but not less than 15 feet from the nearest track. All traffic control devices shall be visible from the driver eye height of 3.50 feet.

D.1.b Sight Triangle

At grade crossings without train activated signal devices, a sight triangle should be provided.

The provision of the capability for defensive driving is an important aspect of the design of rail-highway grade crossings. An early view of an approaching train is necessary to allow the driver time to decide to stop or to proceed through the crossing.

The size of this sight triangle, which is shown in Figure 7 – 1 Visibility Triangle at Rail-Highway Grade Crossings, is dependent upon the train speed limit, the highway design speed, and the highway approach grade. The minimum distance along the highway (d_H), includes the requirements for stopping sight distance, the offset distance (D) from the edge of track to the stopped position (15 feet), and the eye offset (d_e) from the front of vehicles (8 feet); (Figure 7 ~~-~~– 1, Case A). The required distance (d_T) along the track, given in Table 7 – 1 Sight Distance at Rail-Highway Grade Crossings, is necessary to allow a vehicle to stop or proceed across the track safely. Where the roadway is on a grade, the lateral sight distance (d_T) along the track should be increased as noted (Table 7 – 1). This lateral sight distance is desirable at all crossings. In other than flat terrain it may be necessary to rely on speed control signs and devices and to predicate sight distance on a reduced speed of operation. This reduced speed should never be less than 15 mph and preferably 20 mph.

D.1.c Crossing Maneuvers

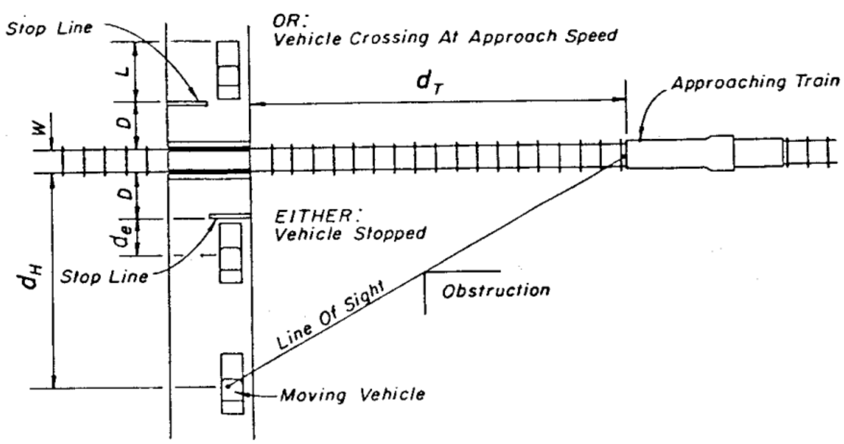
The sight distance required for a vehicle to cross a railroad from a stop is essentially the same as that required to cross a highway intersection as given in ***Chapter 3 – Geometric Design***.

An adequate clear distance along the track in both directions should be provided at all crossings. This distance, when used, shall be no less than the values obtained from Figure 7 – 1 Visibility Triangle at Rail-Highway Grade Crossings and Table 7 – 1 (Case B), Sight Distance at Rail-Highway Grade Crossings. Due to the greater stopping distance required for trains, this distance should be increased wherever possible.

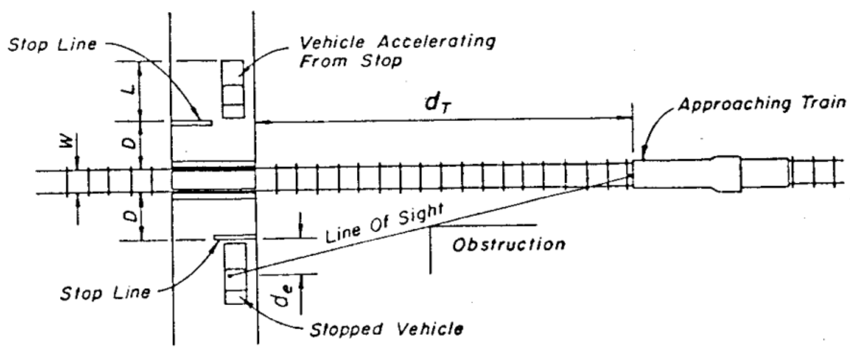
The crossing distance to be used shall include the total width of the tracks, the length of the vehicle, and an initial vehicle offset. This offset shall be at least 10 feet back from any gates or flashing lights, but not less than 15 feet from the nearest track. The train speed used shall be equal to or greater than the established train speed limit.

The setback for determining the required clear area for sight distance should be at least 10 feet more than the vehicle offset. Care should be exercised to ensure signal supports and other structures at the crossing do not block the view of drivers preparing to cross the tracks.

Figure 7 – 1
Visibility Triangle at Rail-Highway Grade Crossings



CASE **A**
APPROACHING VEHICLE TO SAFELY CROSS OR STOP AT RAILROAD CROSSING



CASE **B**
VEHICLE DEPARTING FROM STOPPED POSITION TO SAFELY CROSS RAILROAD TRACK

For d_H and d_T values and crossing conditions see Table ***7-1.***

Table 7 – 1
Sight Distance at Rail-Highway Grade Crossings

Design Sight Distances for Combinations of Train and Highway Vehicle Speeds Conditions:								
Single Track 90° Crossing Design Vehicle WB-62FL and WB-67 (L=73.5’ d_e=8') Flat Highway Grades				Track Width (W) = 5' Vehicle Stop Position (D) = 15' No Train Activated Warning Devices				
Train Speed (mph)	Case B Vehicle Departure From Stop	Case A Moving Vehicle						
	Vehicle Speed (mph)							
	0	10	20	30	40	50	60	70
	d_t (feet) Sight Distance Along Railroad Track							
10	255	155	110	102	102	106	112	119
20	509	310	220	203	205	213	225	239
30	764	465	331	305	307	319	337	358
40	1019	619	441	407	409	426	450	478
50	1274	774	551	509	511	532	562	597
60	1528	929	661	610	614	639	675	717
70	1783	1084	771	712	716	745	787	836
80	2038	1239	882	814	818	852	899	956
90	2292	1394	992	915	920	958	1012	1075
100	2547	1548	1102	1017	1023	1064	1124	1194
110	2802	1703	1212	1119	1125	1171	1237	1314
120	3057	1858	1322	1221	1227	1277	1349	1433
130	3311	2013	1433	1322	1329	1384	1461	1553
(Continued on Next Page)								

Table 7 – 1
Sight Distance at Rail-Highway Grade Crossings
(continued)

d_H (feet) Sight Distance Along Highway							
	69	135	220	324	447	589	751

Notes: 1. Sight distances are required in all quadrants of the crossing.

2. Corrections must be made for conditions other than shown in the table, such as, multiple rails, skewed angle crossings, ascending and descending grades, and curvature of highways and rails. For condition adjustments and additional information refer to Railroad-Highway Grade Crossings under ***Chapter 9*** of "***A Policy on Geometric Design of Highways and Streets", AASHTO (2011)***. Additional information is available on FHWA's website for ***Highway-Rail Grade Crossing Surfaces*** and ***NCHRP Synthesis 250 Highway – Rail Grade Crossing Surfaces, TRB, (1998).***"

Source: Developed from ***Table 9 – 32, A Policy on Geometric Design of Highway and Streets, AASHTO (2011).***

D.2 Approach Alignment

The alignment of the approach roadways is a critical factor in developing a safe grade crossing. The horizontal and vertical alignment, and particularly any combination thereof, should be as gentle as possible.

D.2.a Horizontal Alignment

The intersection of a highway and railroad should be made as near to the right angle (90 degrees) as possible. Intersection angles less than 70 degrees should be avoided. The highway approach should, if feasible, be on a tangent, because the use of a horizontal curve tends to distract the driver from a careful observation of the crossing. The use of superelevation at a crossing is normally not possible, since this would prevent the proper grade intersection with the railroad.

D.2.b Vertical Alignment

The vertical alignment of the roadway on a crossing is an important factor in safe vehicle operation. The intersection of the tracks and the roadway should constitute an even plane. All tracks should, preferably, be at the same elevation, thus allowing a smooth roadway through the crossing. Where the railroad is on a curve with superelevation, the vertical alignment of the roadway shall coincide with the grade established by the tracks.

Vertical curvature on the crossing should be avoided. This is necessary to limit vertical motion of the vehicle.

The vertical alignment of the approach roadway should be adjusted when rail elevations are raised to prevent abrupt changes in grade and entrapment of low clearance vehicles

The roadway approach to crossing should also coincide with the grade established by the tracks. This profile grade, preferably zero, should be extended a reasonable distance (at least two times the design speed in feet) on each side of the crossing. Where vertical curves are required to approach this section, they should be as gentle as possible. The length of these vertical curves shall be of sufficient length to provide the required sight distance.

D.3 Highway Cross Section

Preserving the continuity of the highway cross section through a grade crossing is important to prevent distractions and to avoid hazards at an already dangerous location.

D.3.a Pavement

The full width of all travel lanes shall be continued through grade crossings. The crown of the pavement shall be transitioned gradually to meet the cross sectional grade of the tracks. This pavement cross slope transition shall be in conformance with the requirements for superelevation runoff. The lateral and longitudinal pavement slopes should be designed to direct drainage away from the tracks.

D.3.b Shoulders

All shoulders shall be carried through rail-highway grade crossings without interruption.

The use of full-width paved shoulders is required at all new crossings to maintain a stable surface for emergency maneuvers. The shoulders should be paved a minimum distance of 50 feet on each side of the crossing, measured from the outside rail. It is desirable to pave 100 feet on either side to permit bicycles to exit the travel lane, slow for their crossing, and then make an adequate search before selecting a gap for a return to the travel lane. See ***Chapter 3, Table 3 – 11 Shoulder Widths for Rural Highways*** for further information on shoulder width.

D.3.c Medians

It is recommended that the full median width on divided highways should be continued through the crossing. The median should be contoured to provide a smooth transition on the tracks.

A raised median is the ideal deterrent to discourage motorists from driving around the gates to cross the tracks or making a U-turn prior to the tracks. Flush medians should have channelization devices as a deterrent. Railroad signals and gate assemblies should be installed in the median only when gate arms of 36 feet will not adequately span the approach roadway.

Figure 7 – 2 Flush Median Channelization Devices

Alexander Street, SR 39A, Plant City, FL 1

D.3.d Sidewalks and Shared Use Paths

To provide an accessible route for pedestrians at grade rail-highway crossings, new or existing sidewalks and shared use paths shall be continued across the rail crossing. The surface of the crossing shall be:

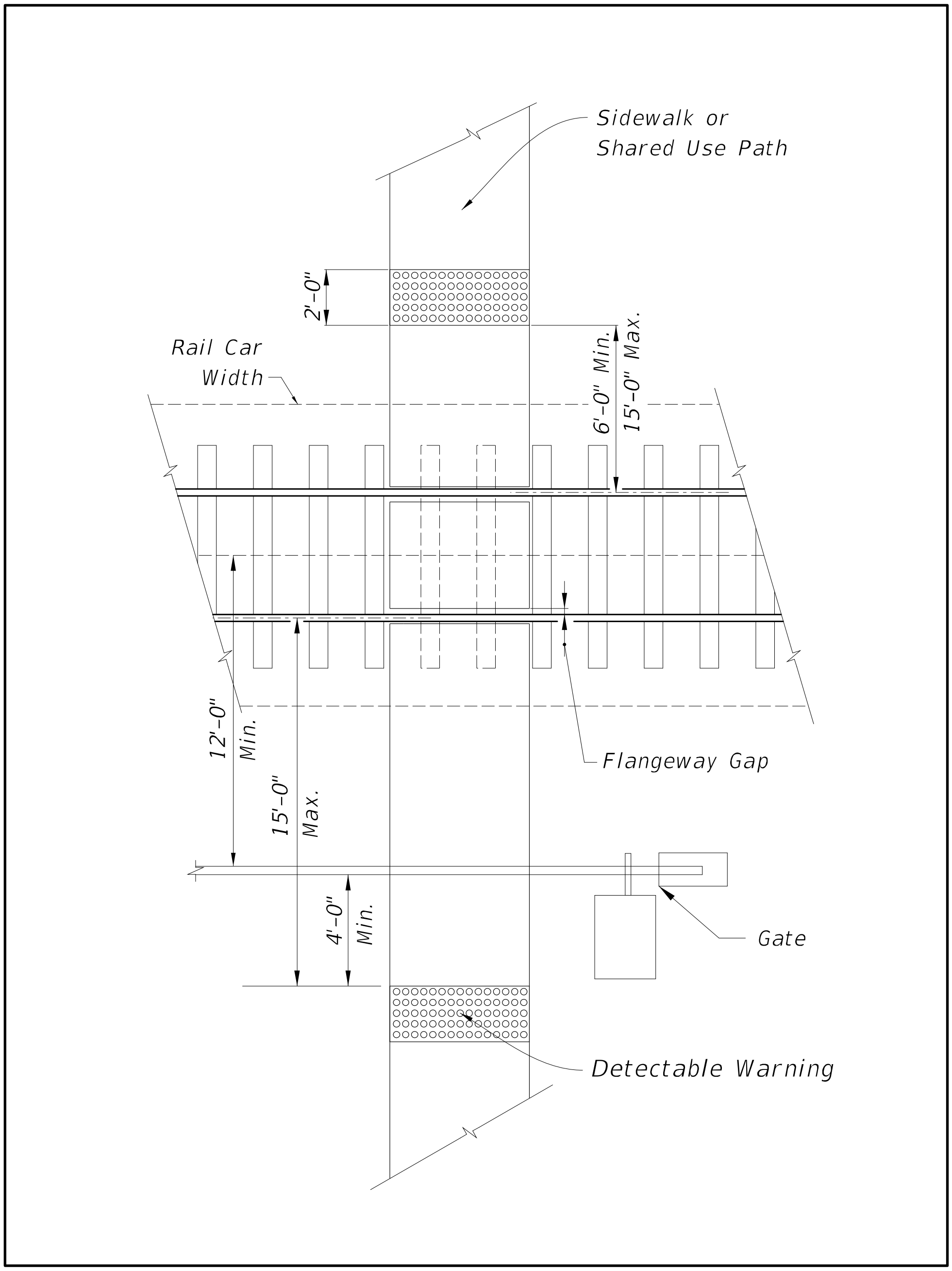
- firm, stable and slip resistant,
- level and flush with the top of rail at the outer edges of the rails, and
- area between the rails align with the top of rail.

Detectable warnings shall be placed on each side of the rail-highway crossing, extend 2.0 feet in the direction of pedestrian travel and the full width across the sidewalk or shared use path, as shown in Figure 7 – 3 Pedestrian Crossings.

The edge of the detectable warning nearest the rail crossing shall be 6.0 to 15.0 feet from the centerline of the nearest rail. Where pedestrian gates are provided, detectable warnings shall be placed a minimum of 4.0 feet from the side of the gates opposite the rail, and within 15.0 feet of the centerline of the nearest rail.

If traffic control signals are in operation at a crossing that is used by pedestrians or bicyclists, an audible device such as a bell shall also be provided and operated in conjunction with the traffic control signals. See ***MUTCD, Chapters 8B and 8C*** for further information and to determine if additional signals, signs, or pedestrian gates should be included. See ***MUTCD, Chapter 8D*** for additional information on designing crossings for shared use paths.

Figure 7 – 3 Pedestrian Crossings

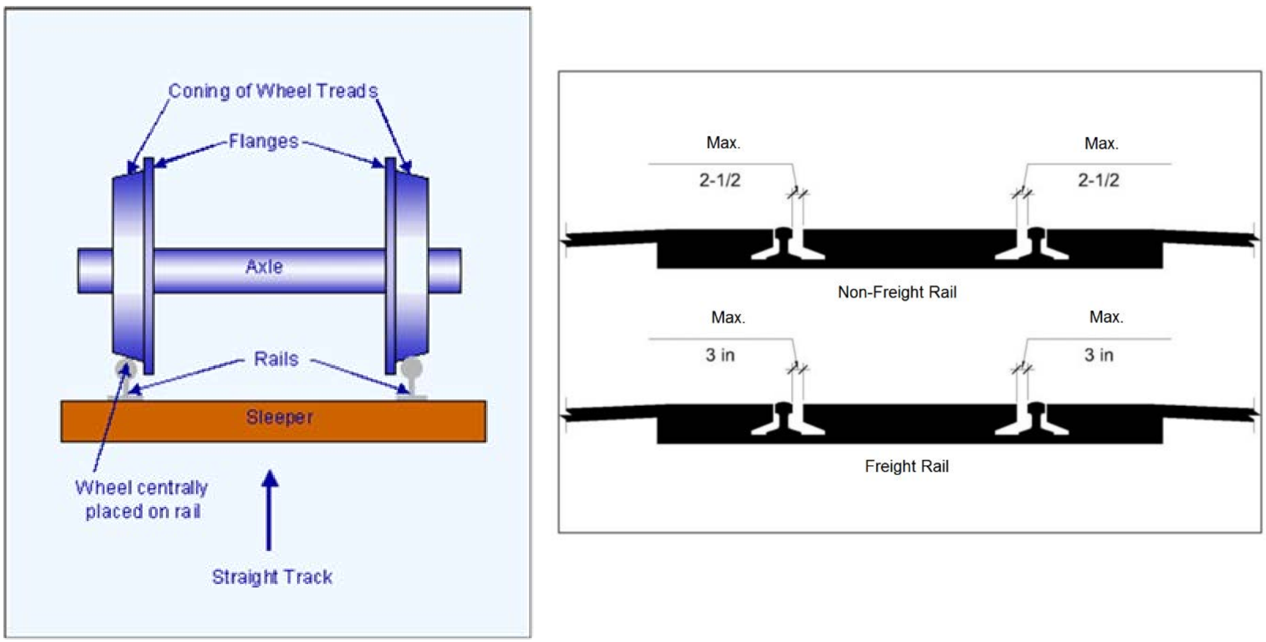


Note: Pedestrian gates may be installed on the outside of the sidewalk/shared use path or in the utility strip.

Flangeway gaps are necessary to allow the passage of train wheel flanges; however, they pose a potential hazard to pedestrians who use wheelchairs because the gaps can entrap the wheelchair casters. Flangeway gaps at pedestrian at-grade rail crossings shall be 2 ½" maximum on non-freight rail track and 3" maximum on freight rail track.

Figure 7 – 4 Flangeways and Flangeway Gaps illustrates where the flanges are located on the wheel, how they interact with the rails, and the maximum gap allowed.

Figure 7 – 4 Flangeways and Flangeway Gaps



See ***Chapter 8 – Pedestrian Facilities*** and ***Chapter 9 – Bicycle Facilities*** for further information on designing sidewalks and shared use paths. The ***2006 Americans with Disabilities Act – Standards for Transportation Facilities*** and the ***2017~~2~~ Florida Accessibility Code*** impose additional requirements for the design and construction of pedestrian facilities.

D.3.e Roadside Clear Zone

Although it is often not practical to maintain the full width of the roadside clear zone, the maximum clear area feasible should be provided. This clear zone shall conform to the requirements for slope and change in grade for roadside clear zones.

D.3.f Auxiliary Lanes

Auxiliary lanes are permitted but not encouraged at signalized rail-highway grade crossings that have a large volume of bus or truck traffic required to stop at all times. These additional lanes should be restricted for the use of these stopping vehicles. The approaches to these auxiliary lanes shall be designed as storage for deceleration lanes. The exits shall be designed as acceleration lanes.

D.4 Roadside Design

The general requirements for roadside design given in ***Chapter 3 – Geometric Design*** and ***Chapter 4 – Roadside Design***, should be followed at rail-highway grade crossings. Supports for traffic control devices may be required within the roadside recovery area. Due to the structural requirements and the necessity for continuous operation, the use of a breakaway design is not recommended. The use of a guardrail or other longitudinal barrier is also not recommended, because an out of control vehicle would tend to be directed into the crossing.

In order to reduce the hazard to errant vehicles, all support structures should be placed as far from the traveled way as practicable.

D.5 Vertical Clearance

Minimum vertical clearances for grade separated rail-highway crossings are shown in Table 7 – 2 Minimum Vertical Clearances for New Bridges. Minimum vertical clearance is the least distance between the bottom of the superstructure and the top of the highest rail utilized anywhere within the horizontal clearance zone.

Table 7 – 2 Minimum Vertical Clearances for New Bridges

Facility Type	Clearance
Railroad over Roadway	16'-6"
Roadway over Railroad[1]	23’-6”
Pedestrian over Railroad[1]	23'-6"

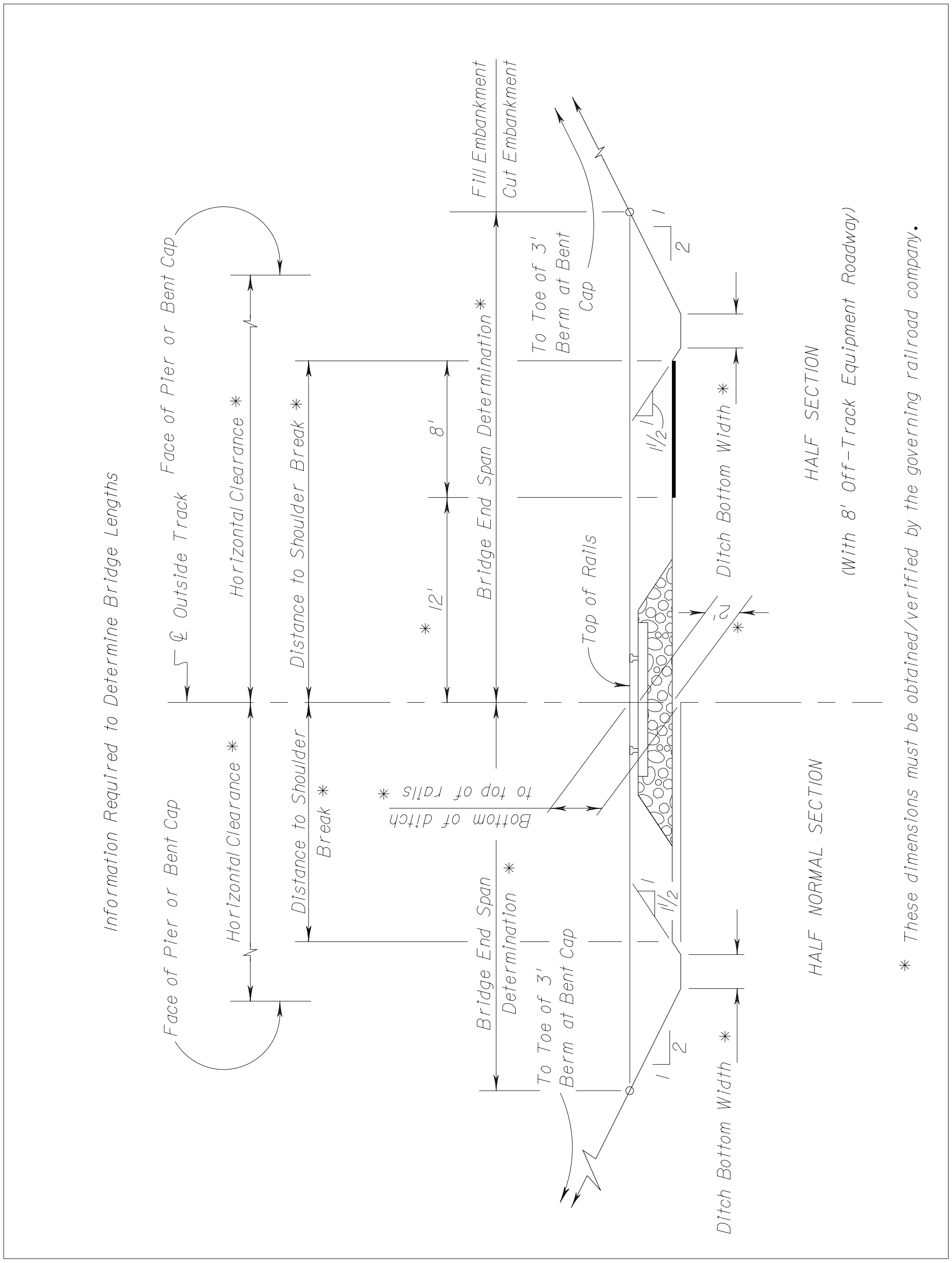
1. Over High Speed Rail Systems, see the latest version of ***American Railway Engineering and Maintenance-of-Way Association (AREMA)*** guidelines, or the design office of the high-speed rail line of interest for specific guidelines and specifications. Over Electrified Railroad, the minimum vertical clearance shall be 24 feet 3 inches. (See ***Department Topic No. 000-725-003: South Florida Rail Corridor Clearance***.)

For any construction affecting existing bridge clearances (e.g., bridge widenings or resurfacing) vertical clearances less than 16' - 0" shall be maintained or increased. If reducing the design vertical bridge clearance to a value between 16’ - 0” and 16' - 2", the design vertical clearance dimension in the plans shall be stated as a minimum.

D.6 Horizontal Clearance

Horizontal clearances shall be measured in accordance with Figure 7 – 5 Track Section. The governing railroad company occasionally may accept a waiver from normal clearance requirements if justified; i.e., for designs involving widening or replacement of existing overpasses. The Department’s District Rail Coordinator should be consulted if such action is being considered for FDOT owned rail corridors. For other rail crossings, coordinate with the owner of the rail corridor.

Figure 7 – 5 Track Section



The minimum horizontal clearances measured from the centerline of outermost existing or proposed tracks to the face of pier cap, bent cap, or any other adjacent structure are shown in Table 7 – 3 Horizontal Clearances for Railroads but must be adjusted for certain physical features and obstructions such as track geometry and physical obstructions.

Table 7 – 3 Horizontal Clearances for Railroads

Minimum Clearance Requirements	Normal Section[1]	With 8' Required Clearance for Off-Track[2]	Temporary Falsework Opening
With Crash Walls	18 ft.	22 ft.	10 ft.
Without Crash Walls	25 ft.	25 ft.	N/A

[1] Any proposed structure over the South Florida Rail Corridor shall be designed and constructed to provide a horizontal clear span of a minimum of 100 feet but not less than 25 feet from the center line of the outermost existing or proposed tracks. (See ***Department Topic No. 000-725-003-j: South Florida Rail Corridor Clearance.***)

[2] The additional 8 ft. horizontal clearance for off-track equipment shall be provided only when specifically requested in writing by the railroad.

D.6.a Adjustments for Track Geometry

When the track is on a curve, the minimum horizontal clearance shall be increased at a rate of 1.5 inches for each degree of curvature. When the track is superelevated, clearances on the inside of the curve will be increased by 3.5 inches horizontally per inch of superelevation. For extremely short radius curves, the AREMA requirements shall be consulted to assure proper clearance.

D.6.b Adjustments for Physical Obstructions

Columns or piles should be kept out of the ditch to prevent obstruction of drainage. Horizontal clearance should be provided to avoid the need for crash walls unless extenuating circumstances dictate otherwise.

Figure 7 – 5 Track Sections shows horizontal dimensions from the centerline of track to the points of intersection of a horizontal plane at the rail elevation with the embankment slope. These criteria may be used to establish the preliminary bridge length, which normally is also the length of bridge eligible for FHWA participation; however, surrounding topography, hydraulic conditions, and economic or structural considerations may warrant a decrease or an increase of these dimensions. These dimensions must be coordinated with the governing railroad company.

The ***Department's Structures Design Guidelines, Section 2.6.7*** provide additional information on the design of structures over or adjacent to railroad and light rail tracks.

D.7 Access Control

The general criteria for access control in ***Chapter 3 – Geometric Design*** for streets and highways should be maintained in the vicinity of rail-highway grade crossings. Private driveways should not be permitted within 150 feet, nor intersections within 300 feet, of any grade crossing.

D.8 Parking

No parking shall be permitted within the required clear area for the sight distance visibility triangle.

D.9 Traffic Control Devices

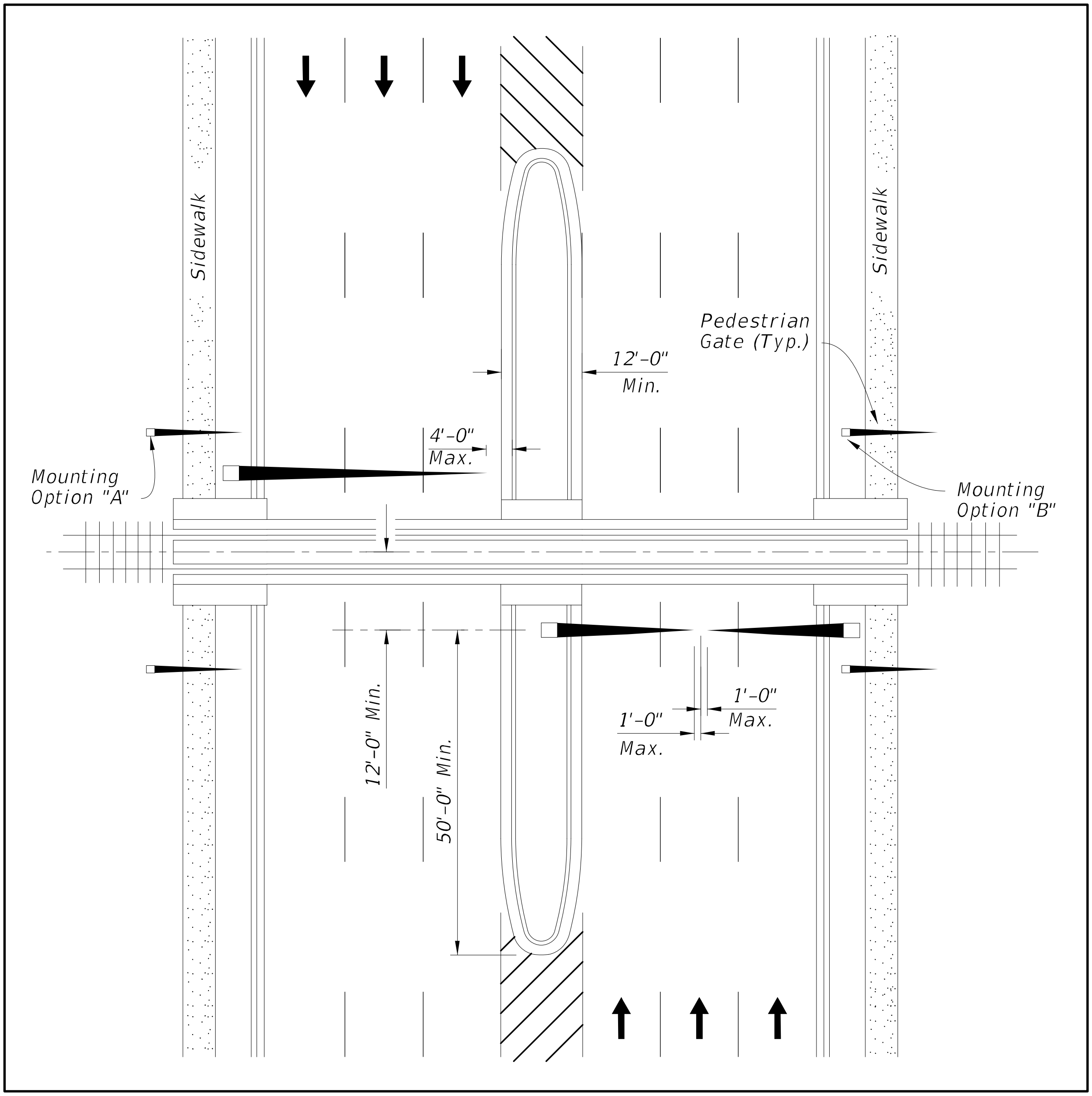
The proper use of adequate advance warning and traffic control devices is essential for all grade crossings. Advance warning should include pavement markings and two or more signs on each approach. Each new crossing should be equipped with train-activated flashing signals.

Automatic gates, when used, should ideally extend across all lanes, but shall at least block one-half of the inside travel lane. It is desirable to include crossing arms across sidewalks and shared use paths.

Traffic control devices shall meet the requirements of the ***MUTCD***. See Section E of this chapter for additional requirements for traffic control devices in Quiet Zones.

Figure 7 – 6 Median Signal Gates for Multilane Curbed Sections provides an example of gate installation when a median is present.

Figure 7 – 6 Median Signal Gates for Multilane Curbed Sections



D.10 Rail-Highway Grade Crossing Surface

Each crossing surface should be compatible with highway user requirements and railroad operations at the site. When installing a new rail-highway crossing or reworking an existing at-grade crossing, welded rail should be placed the entire width from shoulder point to shoulder point. Surfaces should be selected to be as maintenance free as possible.

D.11 Roadway Lighting

The use of roadway lighting at grade crossings should be considered to provide additional awareness to the driver. Illumination of the tracks can also be a beneficial safety aid.

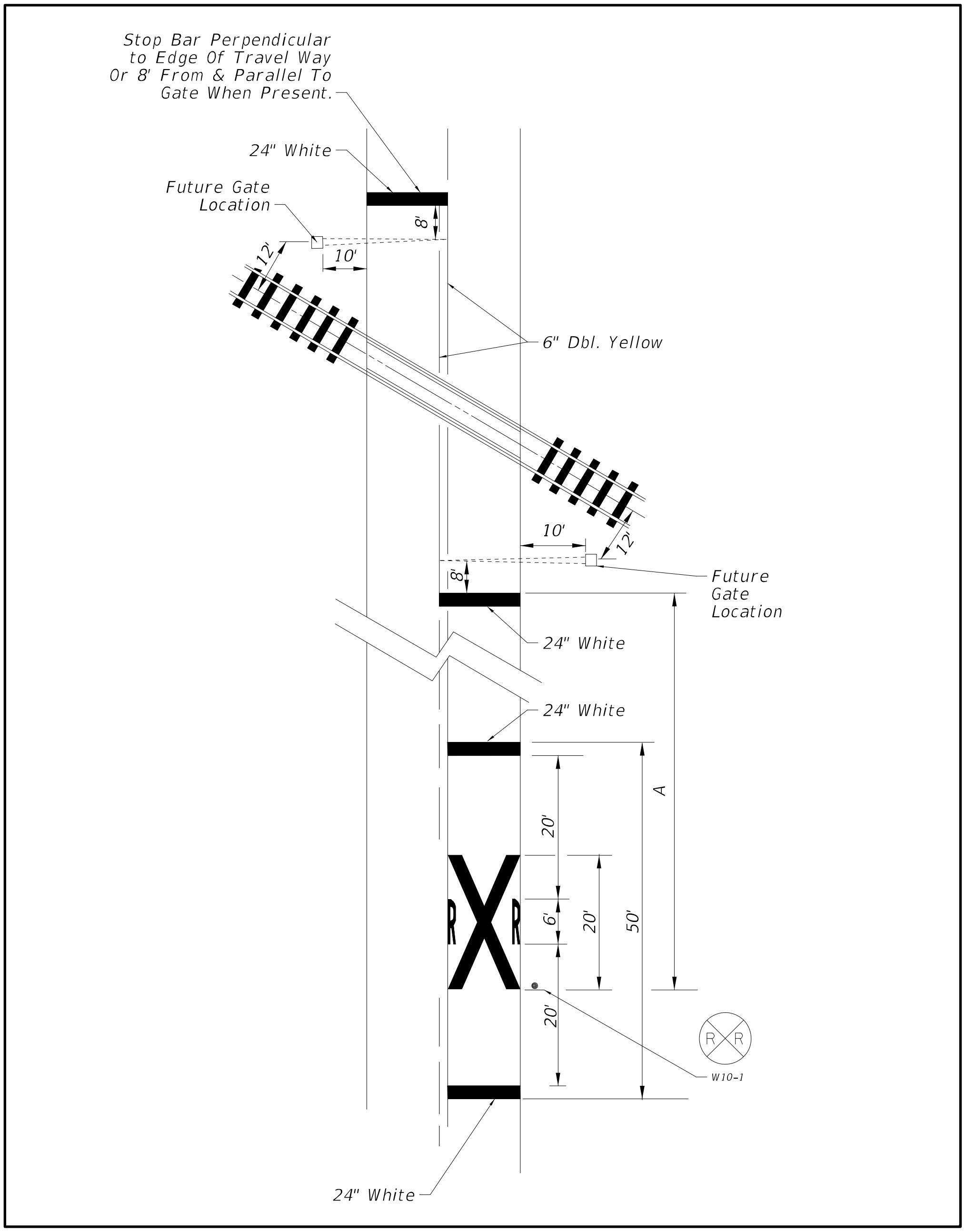
D.12 Crossing Configuration

Recommended layouts for grade crossings are shown in Figures 7 – 7 Passive Rail-Highway Grade Crossing Configuration and 7 – 8 Active Rail-Highway Grade Crossing Configuration. The distance “A” in the Figures is determined by speed and shown in the ***MUTCD, Table 2C – 4. Guidelines for the Advance Placement of Warning Signs***. Although the design of each grade crossing must be "tailored" to fit the existing situation, the principles given in this section should be followed in the design of all crossings. Additional information on the design of rail-highway crossings can be found in the Department’s ***Design Standards, Index 17881 and 17882***.

Passive rail-highway grade crossings include traffic control devices that provide static messages of warning, guidance, and, in some instances, mandatory action for the driver. *(Source: **FHWA Railroad-Highway Grade Crossing Handbook**)*

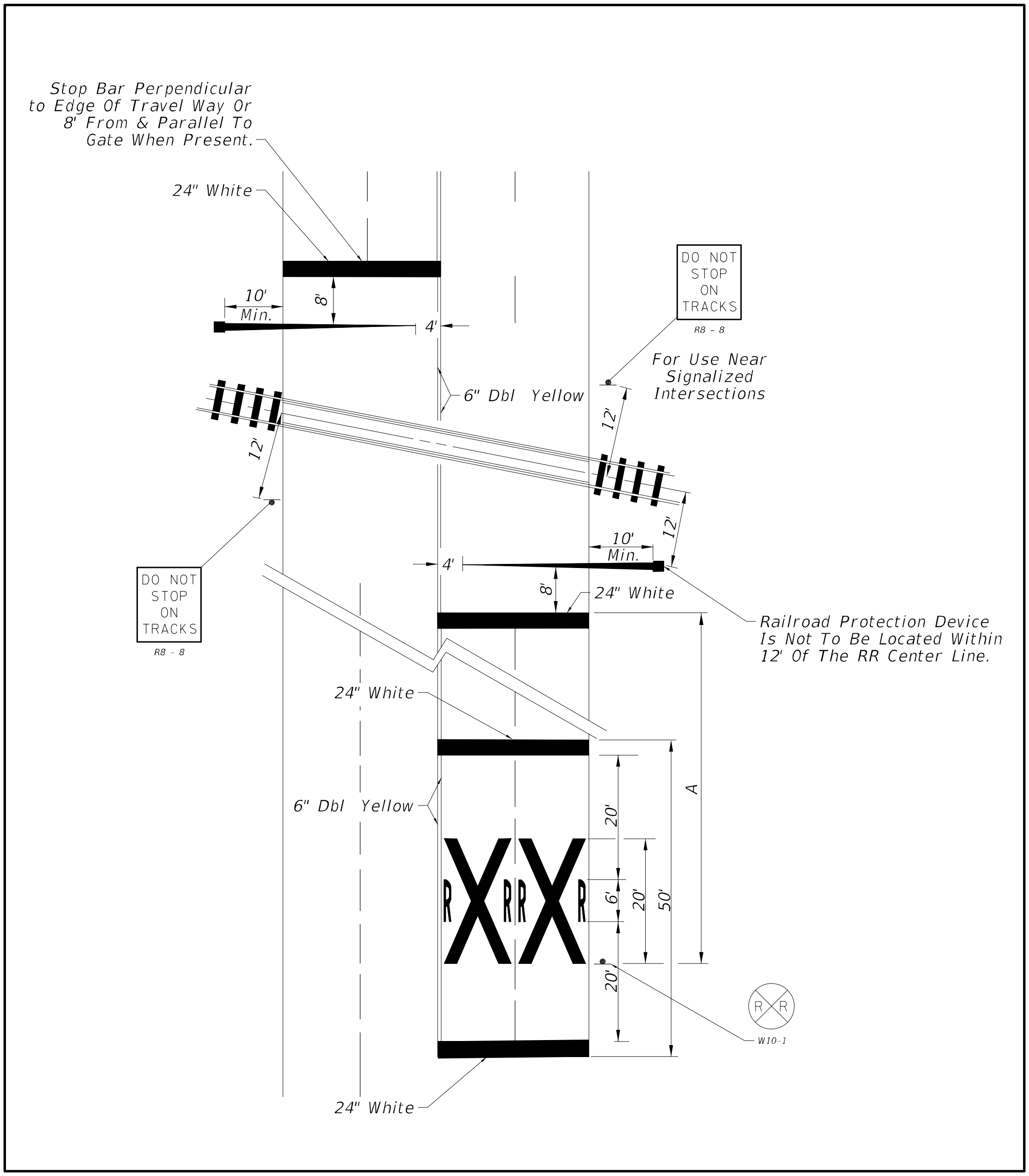
Active rail-highway grade crossings include traffic control devices that give advance notice of the approach of a train. *(Source: **FHWA Railroad-Highway Grade Crossing Handbook**).*

Figure 7 – 7 Passive Rail-Highway Grade Crossing Configuration



Note: The distance "A" is determined by speed and shown in the ***MUTCD, Table 2C – 4. Guidelines for the Advance Placement of Warning Signs***.

Figure 7 – 8 Active Rail-Highway Grade Crossing Configuration



Note: The distance "A" is determined by speed and shown in the ***MUTCD, Table 2C – 4. Guidelines for the Advance Placement of Warning Signs***.

E QUIET ZONES

Quiet Zone means a segment of a rail line that includes public rail-highway crossings at which locomotive horns are not routinely sounded. The Federal Railroad Administration (FRA) has established guidelines the applying jurisdiction must follow for approval of quiet zones. Applying entities can go to the FRA's website and the ***Code of Federal Regulations (CFR), Title 49, Subtitle B, Chapter II, Part 222*** for further information on the process for approval of Quiet Zones.

Coordinate with the Department's District Rail Coordinator to determine if crossings are located within designated Quiet Zones for State owned rail corridors or crossings of state highways. State owned rail corridors include the ***Central Florida Rail Corridor*** and ***South Florida Rail Corridor***. For other rail crossings, coordinate with the local government who maintains the crossing roadway, sidewalk or shared use path to determine if the location has been approved by the FRA for a Quiet Zone.

For a crossing within a Quiet Zone that requires supplemental safety measures, approved supplemental safety measures include:

- Temporary closure of a public railroad-highway-rail grade crossing;
- Four-quadrant gate systems;
- Gates with medians or channelization devices;
- One way street with gate(s); and
- Permanent closure of a public highway-rail grade crossing.

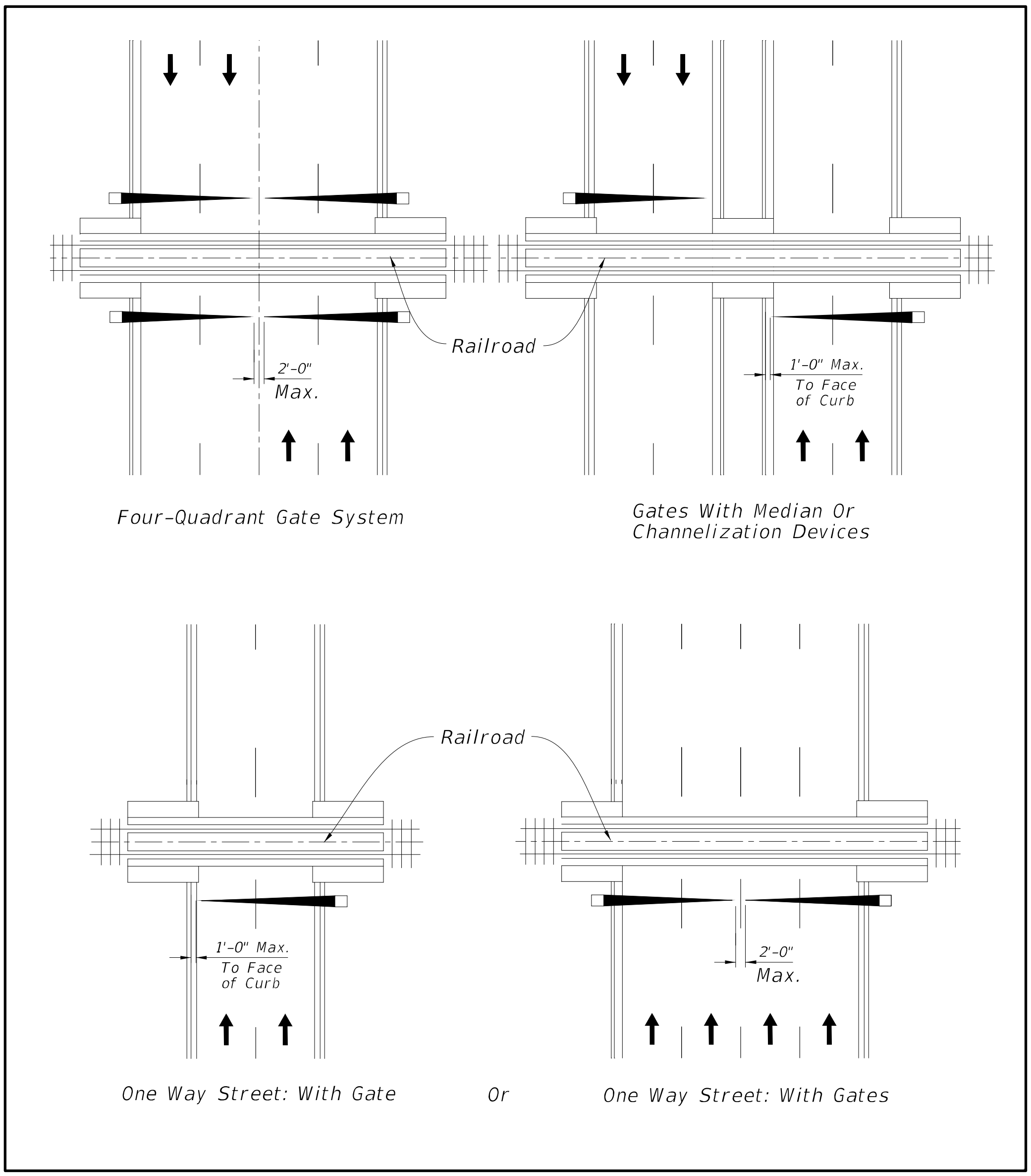
The ***CFR, Title 49, Chapter II, Part 222, Appendix A, Approved Supplemental Safety Measures*** provides additional information on the design of Quiet Zones to meet federal approval. The ***CFR*** also requires that any traffic control device and its application where used as part of a Quiet Zone shall comply with all applicable provisions of the ***MUTCD.*** See ***MUTCD, Part 8, Traffic Control for Railroad and Light Rail Transit Grade Crossings*** for further information. Pedestrian gates, audible device, and detectable warnings are required when a sidewalk or shared use path is present or proposed.

For Quiet Zones that cross state owned rail corridors, the Department's ***Design Manual, Chapter 220 Railroads*** ~~***Plans Preparation Manual, Volume 1, Chapter***~~

~~6~~ provides additional design criteria.

Figure 7 – 9 Gate Configurations for Quiet Zones illustrates the maximum gap allowed for gates at rail-highway crossings within Quiet Zones, based upon ***CFR, Title 49, Chapter II, Part 222.***

Figure 7 – 9 Gate Configuration for Quiet Zones



F HIGH SPEED RAIL

The establishment of high-speed rail service is governed by ***49 U.S. Code 26106 – High-Speed Rail Corridor Development***.

The ***High-Speed Rail (HSR) Strategic Plan*** divides potential operations into four categories or generic descriptions:

- HSR – Express. Frequent express service between major population centers 200 - 600 miles apart, with few intermediate stops. Top speeds of at least 150 mph on completely grade-separated, dedicated rights-of-way (with the possible exception of some shared track in terminal areas). Intended to relieve air and highway capacity constraints.

- HSR – Regional. Relatively frequent service between major and moderate population centers 100 - 500 miles apart, with some intermediate stops. Top speeds of 110 - 150 mph, grade-separated, with some dedicated and some shared track (using positive train control (PTC) technology). Intended to relieve highway and, to some extent, air capacity constraints.

- Emerging HSR. Developing corridors of 100 - 500 miles, with strong potential for future HSR Regional and/or Express service. Top speeds of up to 80 - 110 mph on primarily shared track (eventually using PTC technology), with advanced grade crossing protection or separation. Intended to develop the passenger rail market and provide some relief to other modes.

- Conventional Rail. Traditional intercity passenger rail services of more than 100 miles with as little as 1 to as many as 7 - 12 daily frequencies; may or may not have strong potential for future high-speed rail service. Top speeds of up to 79 mph generally on shared track. Intended to provide travel options and to develop the passenger rail market for further development in the future.

Further information on the implementation of high-speed rail service can be found on the Federal Railroad Administration's website ***High Speed Rail Overview***.

G MAINTENANCE AND RECONSTRUCTION

The inspection and maintenance of all features of rail-highway grade crossings shall be an integral part of each highway agency's and railroad company's regular maintenance program (***Chapter 10 – Maintenance And Resurfacing***). Items that should be given a high priority in this program include: pavement stability and skid resistance, clear sight distance, and all traffic control and protective devices.

The improvement of all substandard or hazardous conditions at existing grade crossings is extremely important and should be incorporated into the regular highway reconstruction program. The objective of this reconstruction program should be to upgrade each crossing to meet these standards. The priorities for reconstruction should be based upon the guidelines set forth by the Department.

H REFERENCES

The following is a list of publications that for further guidance:

- Federal Highway Administration Railroad-Highway Grade Crossing Handbook, Revised Second Edition, August 2007
  http://safety.fhwa.dot.gov/xings/com_roaduser/07010/

- Code of Federal Regulations (CFR), Title 49 Transportation, Part 222, Use of Locomotive Horns at Public Highway-Rail Grade Crossings
  http://www.ecfr.gov/cgi-bin/text-idx?tpl=/ecfrbrowse/Title49/49cfr222_main_02.tpl

- The Train Horn Rule and Quiet Zones
  https://www.fra.dot.gov/Page/P0104

- MUTCD, Part 8, Traffic Control for Railroad and Light Rail Transit Grade Crossings
  http://mutcd.fhwa.dot.gov/pdfs/2009r1r2/part8.pdf

- The American Railway Engineering and Maintenance-of-Way Association (AREMA)
  https://www.arema.org/

- Florida Administrative Code, (Rule 14-57: Railroad Safety and Clearance Standards, and Public Railroad-Highway Grade Crossings
  https://www.flrules.org/gateway/RuleNo.asp?title=RAILROAD SAFETY AND CLEARANCE STANDARDS, AND PUBLIC RAILROAD-HIGHWAY GRADE CROSSINGS&ID=14-57.011

- Florida Department of Transportation Rail Contacts
  http://www.dot.state.fl.us/rail/contacts.shtm

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