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 – 264 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 1~~2~~ ~~-~~– Planning and 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, 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 – 19***~~6~~ 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 – 25 Minimum Stopping Sight Distance (Rounded Values). These values are applicable for any street, highway, or turning roadway. The driver should, at this required distance, have a clear view of the intersecting roadway, as well as the sign or traffic signal.

Where the approach roadway is on a grade or vertical curve, the sight distance should be no less than the values shown in ***Figure 3 ~~-~~– 18~~5~~*** 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.

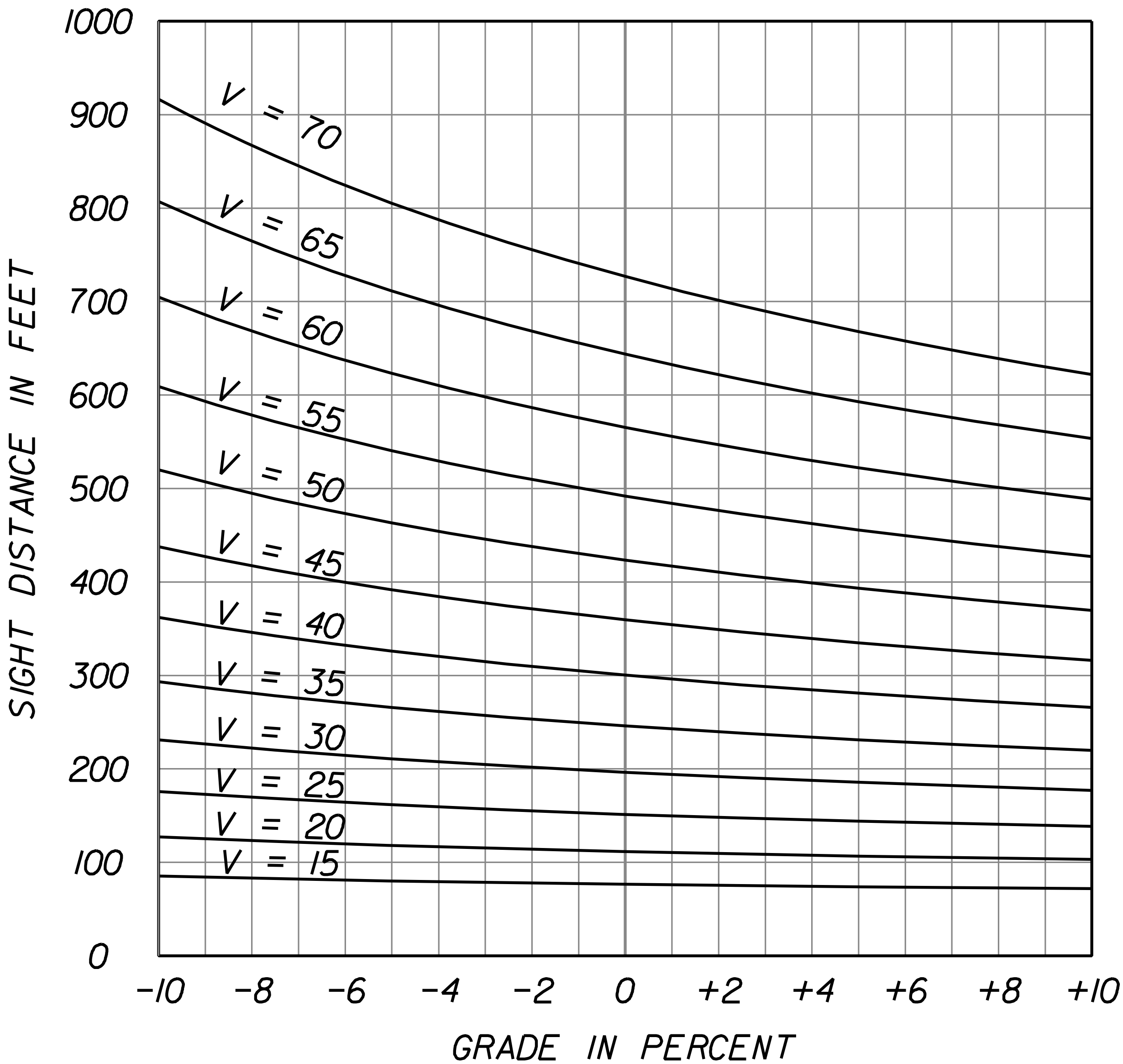
Table 3 – 27 Minimum Stopping Sight Distance (Rounded Values)

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

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 – 27~~5~~*** Minimum Stopping Sight Distance (Rounded Values) or ***Figure 3 – 18~~5~~*** Sight Distances for Approach to Stop on Grades. Due to the inability of vehicle headlights to adequately illuminate a sharply curved travel path, roadway lighting should be considered for turning roadways.

Figure 3 – 18~~5~~ 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 ~~-~~– 19~~6~~ Departure Sight Triangle (Traffic Approaching from Left or Right) and 3 – 20~~17~~ 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:

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

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 – 19~~16~~ Departure Sight Triangle
(Traffic Approaching from Left or Right)

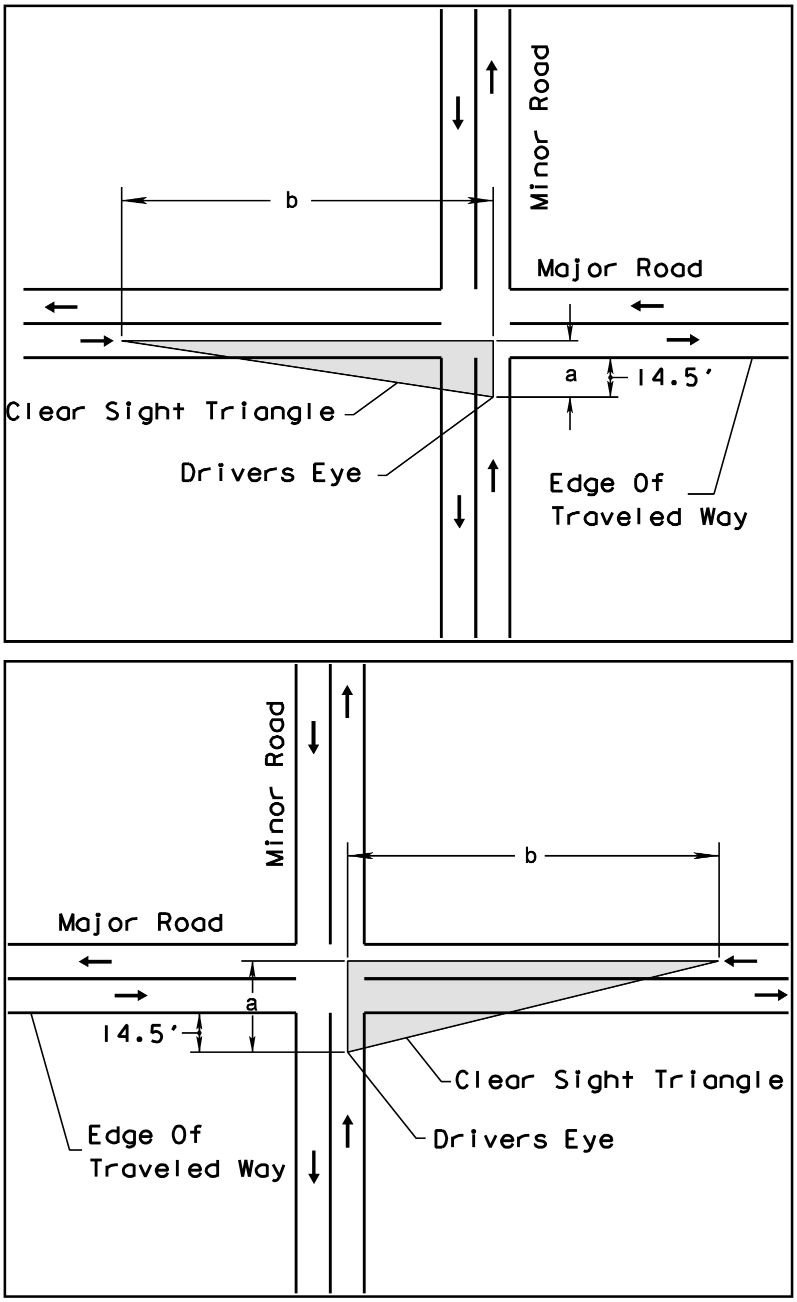
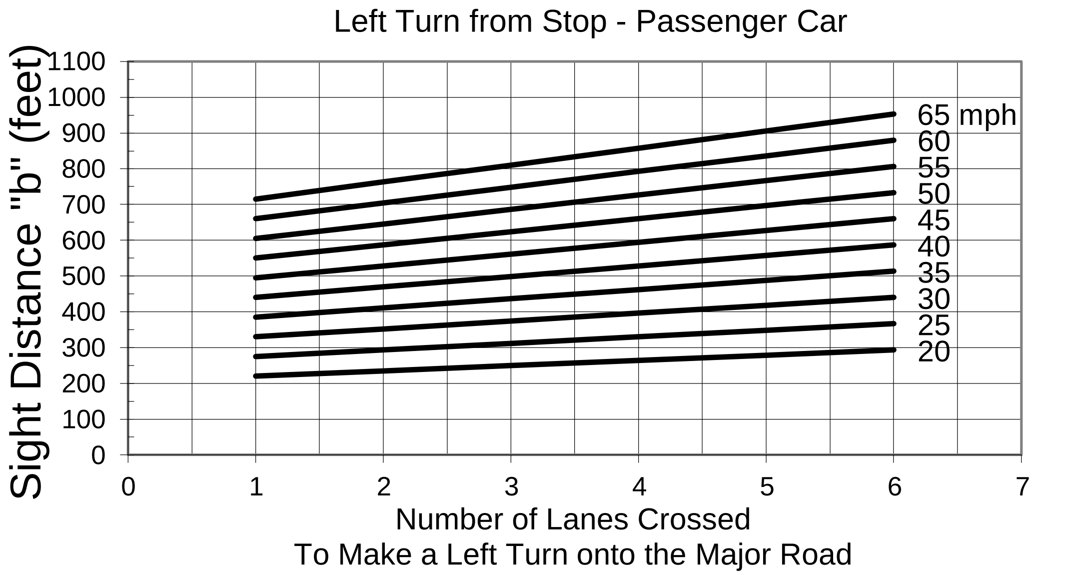
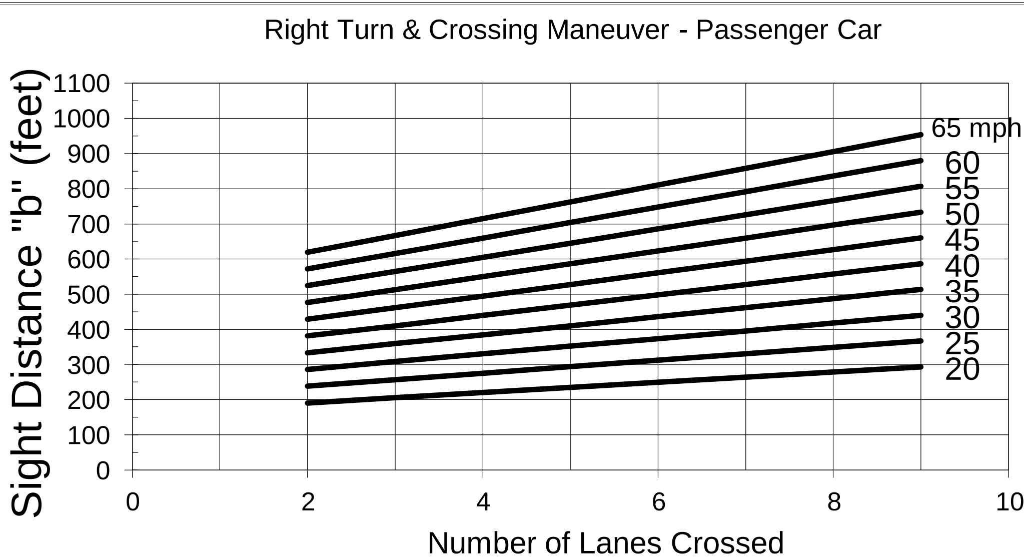


Figure 3 – 20~~17~~ Intersection Sight Distance



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 – 21~~18~~ Sight Distance for Vehicle Turning Left from Major Road).

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

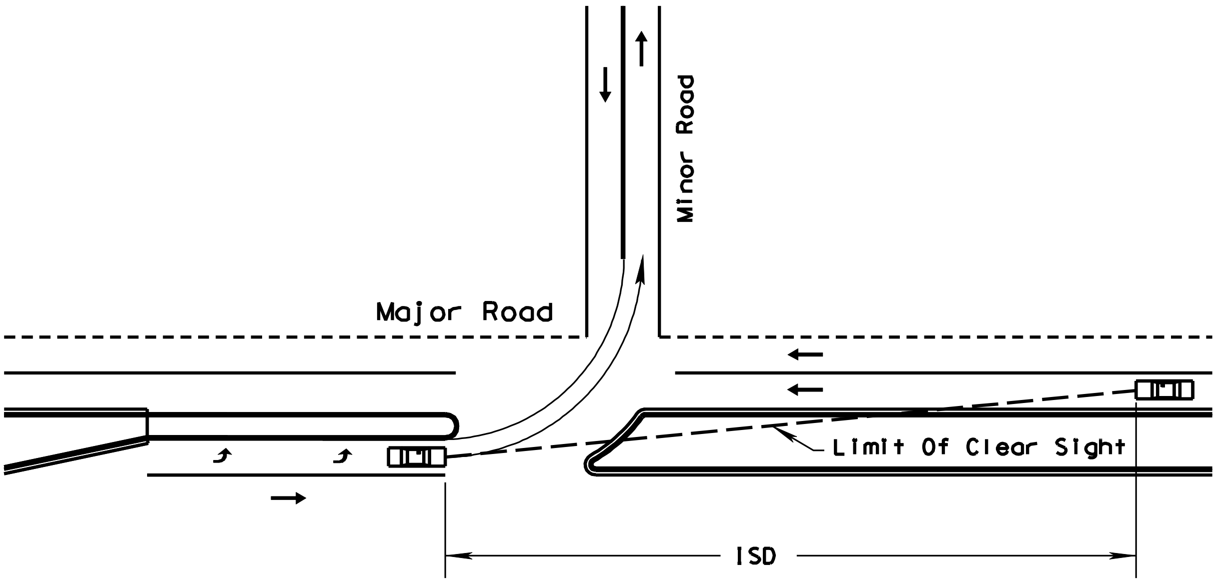
Design Vehicle	Time Gap (t_g) in Seconds
Passenger Car	5.5
Single Unit Truck	6.5
Combination Truck	7.5

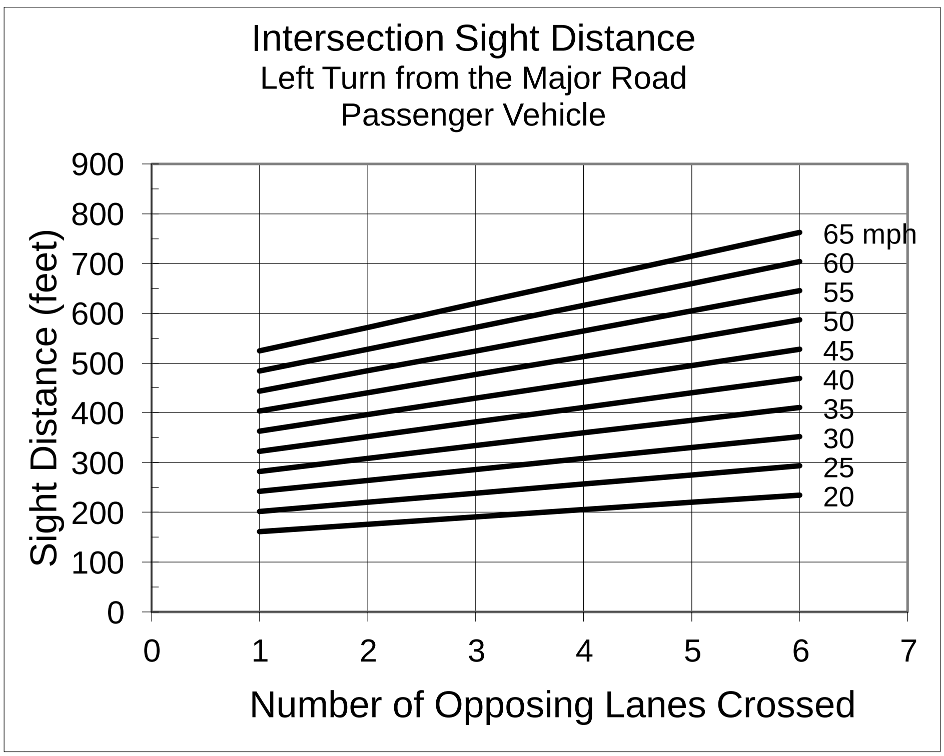
For left turning vehicles that cross more than one opposing lane, add 0.5 seconds for passenger cars and 0.7 seconds for trucks for each additional lane to be crossed.

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

The~~The Department~~ ***FDOT 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 (2011)***.

Figure 3 – 2118 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 – 20 Minimum Lane Widths.

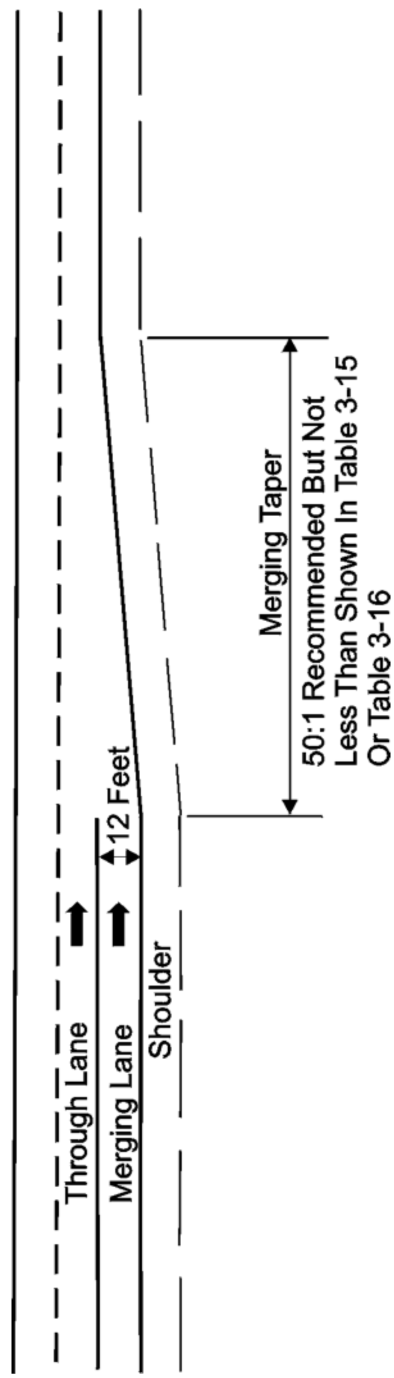
C.9.c.1 Merging Maneuvers

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 – 28~~6~~ 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 –~~-~~ 22~~19~~ 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 – 28~~6~~ 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 – 22~~19~~ 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 – 29~~7~~ Design Lengths of Speed Change Lanes Flat Grades. Where acceleration occurs on a grade, the required distance is obtained by using Table~~s~~ 3 ~~-~~– 30~~28~~ Ratio of Length of Speed Change Lane on Grade to Length on Level and Table 3 ~~-~~– 31~~29~~ Minimum Acceleration Lengths for Entrance Terminals.

The final speed at the end of the acceleration lane, should, desirably, be assumed as the design speed of the through roadway. The length of acceleration lane provided should be at least as long as the distance required for acceleration between the initial and final speeds. Due to the uncertainties regarding vehicle capabilities and driver behavior, additional length is desirable. The acceleration lane should be followed by a merging taper (similar to Figure 3 – 23~~19~~ Termination of Merging Lanes), not less than that length set forth in Table 3 – 28 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 – 22~~19~~ Termination of Merging Lanes. Recommended acceleration lanes for freeway entrance terminals are given in Table 3 – 29 Minimum Acceleration Lengths for Entrance Terminals.

Table 3 – 29~~7~~ 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.

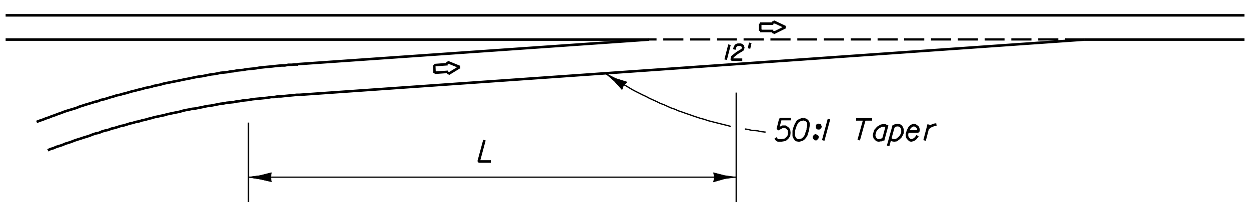
Table 3 – 30~~28~~ Ratio of Length of Speed Change Lane on Grade to Length on Level

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 – 26 give the length of speed change lane for the respective grade.								

Table 3 – 31~~29~~ Minimum Acceleration Lengths for Entrance Terminals

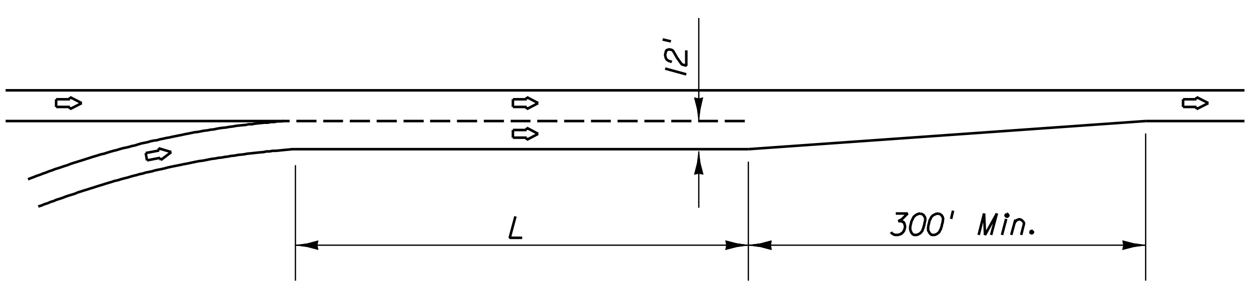
Highway Design Speed (mph)	L = Acceleration Length (feet)								
	For Entrance Curve Design Speed (mph)								
	Stop Condition	15	20	25	30	35	40	45	50
30	180	140	---	---	---	---	---	---	---
35	280	220	160	---	---	---	---	---	---
40	360	300	270	210	120	---	---	---	---
45	560	490	440	380	280	160	---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

The required distance for deceleration on grades is given in Tables 3 – 29~~7~~ Design Lengths of Speed Change Lanes Flat Grades - 2 Percent or Less and 3 – 30~~28~~ Ratio of Length of Speed Change Lane on Grade to Length on Level.

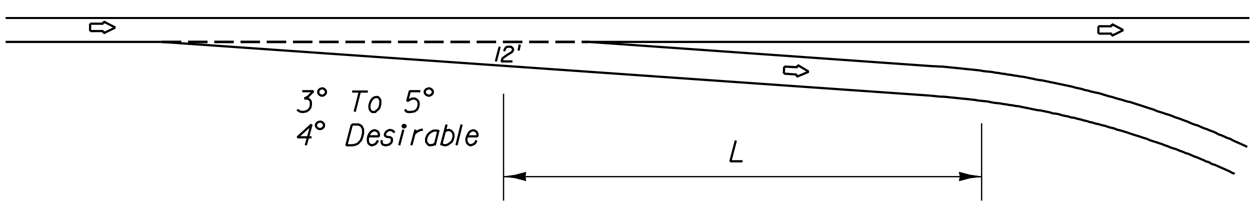
The length of deceleration lanes shall be no less than the values obtained from Tables 3 – 29~~7~~ and 3 – 30~~28~~ and should be increased wherever feasible. The initial speed should, desirably, be taken as the design speed of the highway. The final speed should be the design speed at the exit (e.g., 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 – 23~~0~~ Entrance for Deceleration Lane. The initial length of straight taper, shown in Table 3 – 31~~29~~ Minimum Acceleration Lengths for Entrance Terminals, may be utilized as a portion of the total required deceleration distance. The pavement surface of the deceleration lane should be clearly visible to approaching traffic, so drivers are aware of the maneuvers required. Recommended deceleration lanes for exit terminals are given in Table 3 – 32~~0~~ Minimum Deceleration Lengths for Exit Terminals.

Table 3 – 32~~0~~ Minimum Deceleration Lengths for Exit Terminals

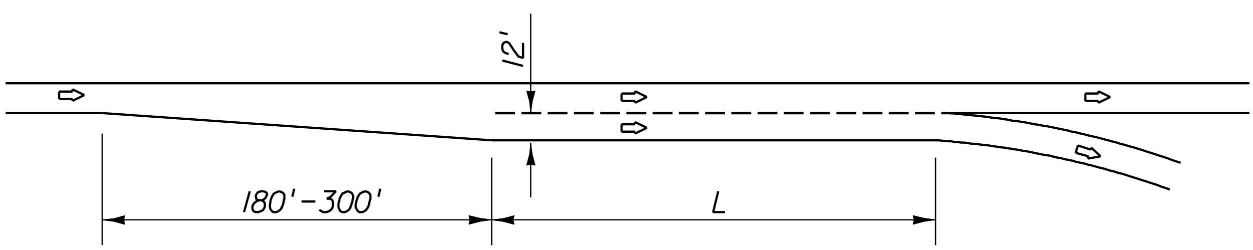
Highway Design Speed (mph)	L = Deceleration Length (feet)								
	For Design Speed of Exit Curve (mph)								
	Stop Condition	15	20	25	30	35	40	45	50
30	235	200	170	140	---	---	---	---	---
35	280	250	210	185	150	---	---	---	---
40	320	295	265	235	185	155	---	---	---
45	385	350	325	295	250	220	---	---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

Expressway and Freeway Exit Terminals



TAPER TYPE

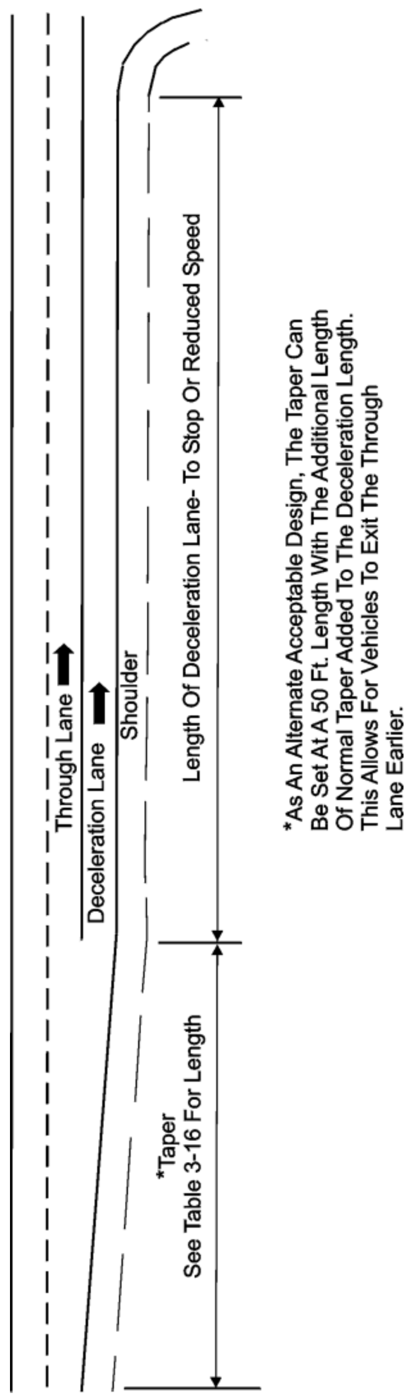
Recommended when design speed at exit curve is 50 mph or greater and when approach visibility is good.



PARALLEL TYPE

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

Figure 3 – 23~~0~~ Entrance for Deceleration Lane



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 – 20 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 – 24 Auxiliary Lanes for Deceleration at Intersections (Turn Lanes) and Table 3 – 33 Turn Lanes – Curbed and Uncurbed Medians. These lengths are based on ~~the Department~~FDOT criteria. As shown in Figure 3 – 24~~1,~~ 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 – 24 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 Departments~~ The FDOT's criteria for approach end taper lengths for turn lanes are 50 feet for a single turn lane and 100 feet for a double turn lane, as shown in Figure 3 - 24~~21~~ Auxiliary Lanes for Deceleration at Intersections (Turn Lanes) and Table 3 – 33 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 – 24~~1~~ Auxiliary Lanes for Deceleration at Intersections (Turn Lanes)

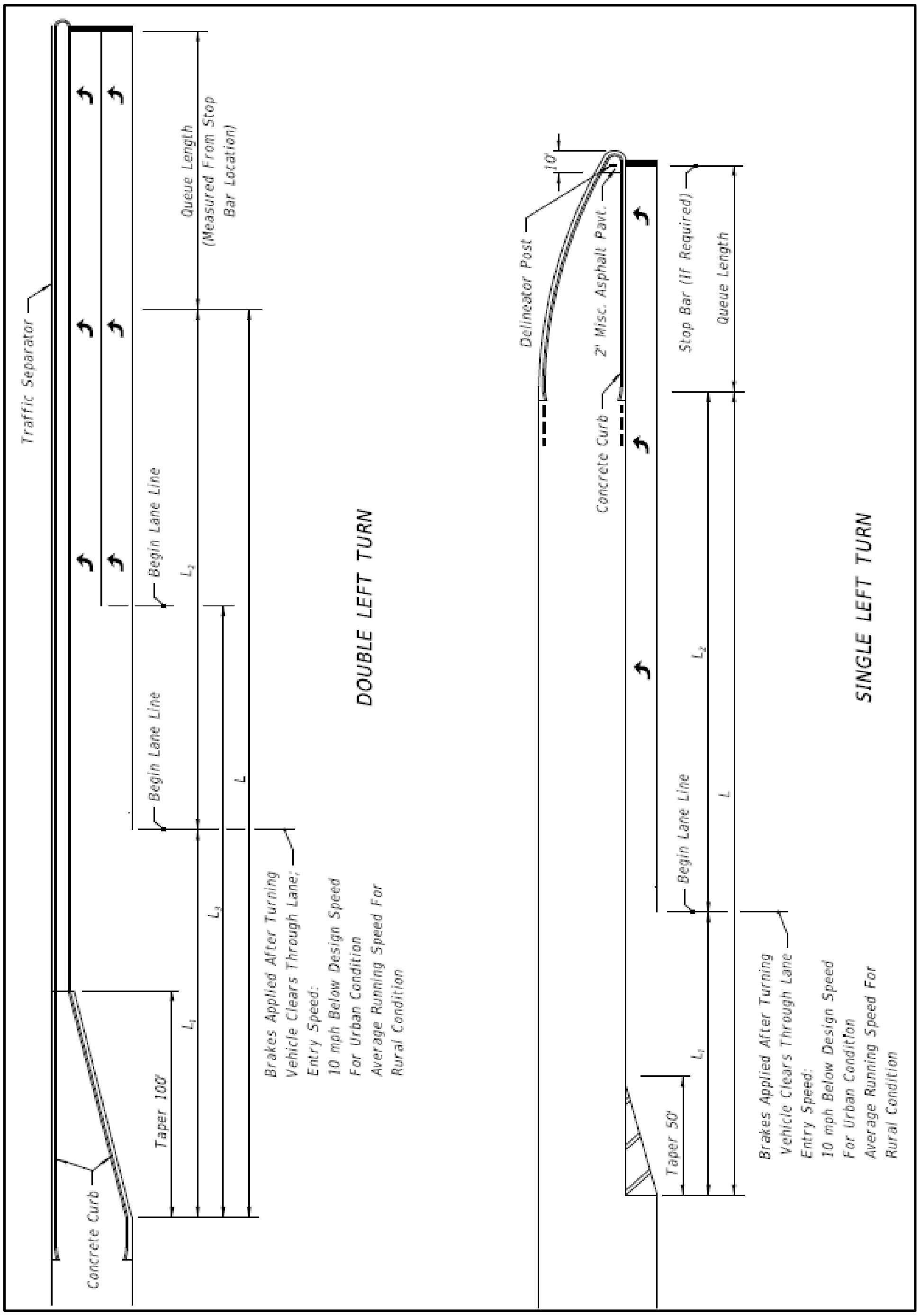


Table 3 – 33~~1~~ Turn Lanes – Curbed and Uncurbed Medians

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

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

C.9.c.4.(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 – 34~~2~~ 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 – 34~~2~~ 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 – 35~~3~~ Maximum Rate of Change in Pavement Edge Elevation for Curves at Intersections and 3 – 36~~4~~ 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 – 354 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 – 364 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 – 4 Minimum Stopping Sight Distances. For additional guidance on vertical alignment for turning roadways, see ***AASHTO – "A Policy on Geometric Design of Highways and Streets" - 2011***.

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 - 37~~5~~ 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 – 37~~5~~ Derived Pavement Widths for Turning Roadways for Different Design Vehicles

Radius on Inner Edge of Pavement, R (feet)	Case 1, One-Lane Operation, No Provision for Passing a Stalled Vehicle												
	P	**SU-30**	**Su-40**	**City Bus**	**S-Bus-36**	**A-Bus**	**WB-40**	**WB-62**	**WB-67**	**WB-67D**	**MH**	**P/T**	**P/B**
50	13	18	21	21	18	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

Table 3 – 37~~5~~ Derived Pavement Widths for Turning Roadways for Different Design Vehicles (Con't)

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

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 – 32~~0~~ Minimum Deceleration Lengths for Exit Terminals. Exits should be on the right and should be placed on horizontal curves. Where deceleration on an exit loop is required, the deceleration alignment should be designed so the driver receives adequate warning of the approaching increase in curvature. This is best accomplished by gradually increasing the curvature and the resulting centrifugal force. This increasing centrifugal force provides warning to the driver that he must slow down. A clear view of the exit loop should also be provided. The length of deceleration shall be no less than the values shown in Table – 29.

Entrances to freeways should be designed in accordance with the general configurations shown below Table 3 – 31~~29~~ Minimum Acceleration Lengths for Entrance Terminals. Special care should be taken to ensure vehicles entering from loops are not directed across through travel lanes. The entering roadway should be brought parallel (or nearly so) to the through lanes before entry is permitted. Where acceleration is required, the distances shown in Table 3 – 31~~29~~ shall, as a minimum, be provided. Exits and entrances to all high-speed facilities (design speed greater than 50 mph), should, where feasible, be designed in accordance with Tables 3 – 32~~0~~ and 3 – 31~~29~~. The lengths obtained from Tables 3 – 32~~0~~ and 3 –31~~29~~ should be adjusted for grade by using the ratios in Table 3 – 30~~28~~.

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.

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

C.10.a.2 Accessibility Requirements

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

C.10.a.3 Sidewalks and Shared Use Paths

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

Bicycle Facilities. ~~Section C.7.d of this chapter~~. To ensure compliance with federal and state accessibility requirements for sidewalks:

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

~~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 1:120 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*** , 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 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 turnaround area, considering backing movements, which will accommodate single truck or transit vehicles without encroachment upon private property. Recommended treatment for dead end streets and cul-de-sacs is given in Figure 5-1 Types of Cul-de-Sacs and Dead-End Streets of ***AASHTO – "A Policy on Geometric Design of Highways and Streets" - 2011***.

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 in flush shoulder sections. See ***Chapter 4 – Roadside Design***, Table 4 – 2 Lateral Offset for further information.

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

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

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 and Variations*** 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 – 20 Minimum Lane Widths 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

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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. 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 ~~the~~ FDOT's ~~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. The intent of this chapter is to reduce the consequences of crashes by vehicles leaving the roadway. The design of the roadside beyond the shoulder should be considered and conducted as an integral part of the total highway design.

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.
- Providing 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 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

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 shall follow the requirements for clear zone and lateral offset shown in this manual.~~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. Clear zone is measured from the edge of the traveled way.

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 ***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]
(Curbed and Flush Shoulder Roadways)

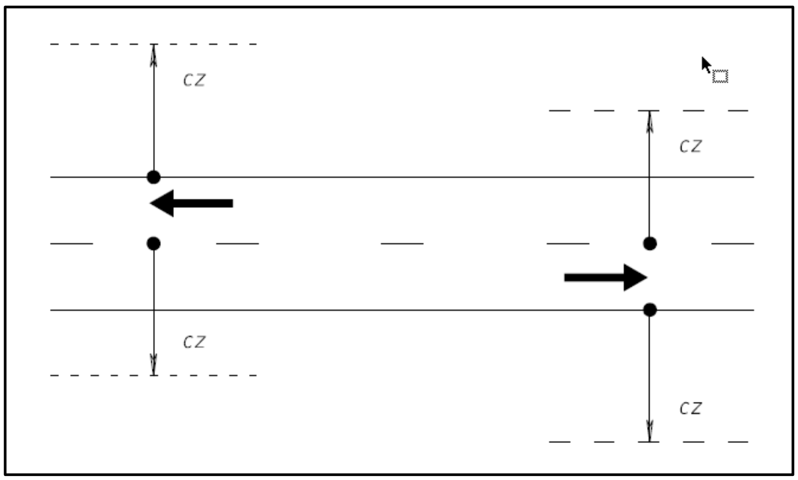
Design Speed mph	AADT ≥ 1500			AADT < 1500 ~~1, 2~~		
	Travel Lanes & Multilane Ramps		Aux Lanes and Single Lane Ramps	Travel Lanes & Multilane Ramps		Aux Lanes and Single Lane Ramps
	1V:6H or flatter	1V:5H to 1V:4H	1V:4H or flatter	1V:6H or flatter	1V:5H to 1V:4H	1V:4H or flatter
≤ 40	14	16	10	10[2]	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.

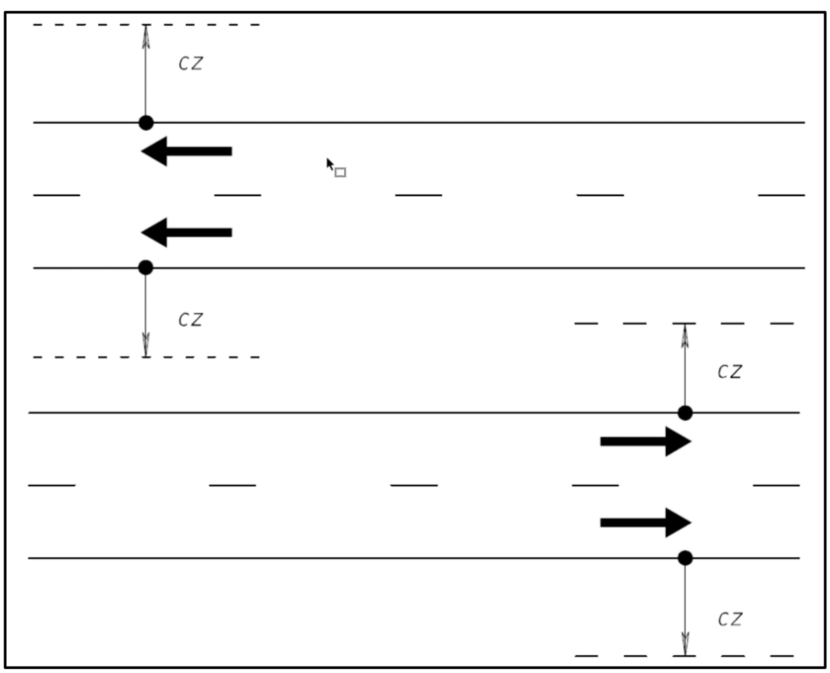
Source: Table 3 – 1, Suggested Clear Zone Distances in Feet from the Edge of the Travel Lane, 2011 AASHTO Roadside Design Guide.

Figure 4 – 1 Clear Zone Plan View

Two Lane, Two -Way Roadway



Multi-Lane Two-Way Roadway



Note: 1. Lateral offset is measured out from the centerline of roadway and edge of traveled way or face of curb to a roadside object or feature.

Figure 4 – 2 Basic Clear Zone Concept

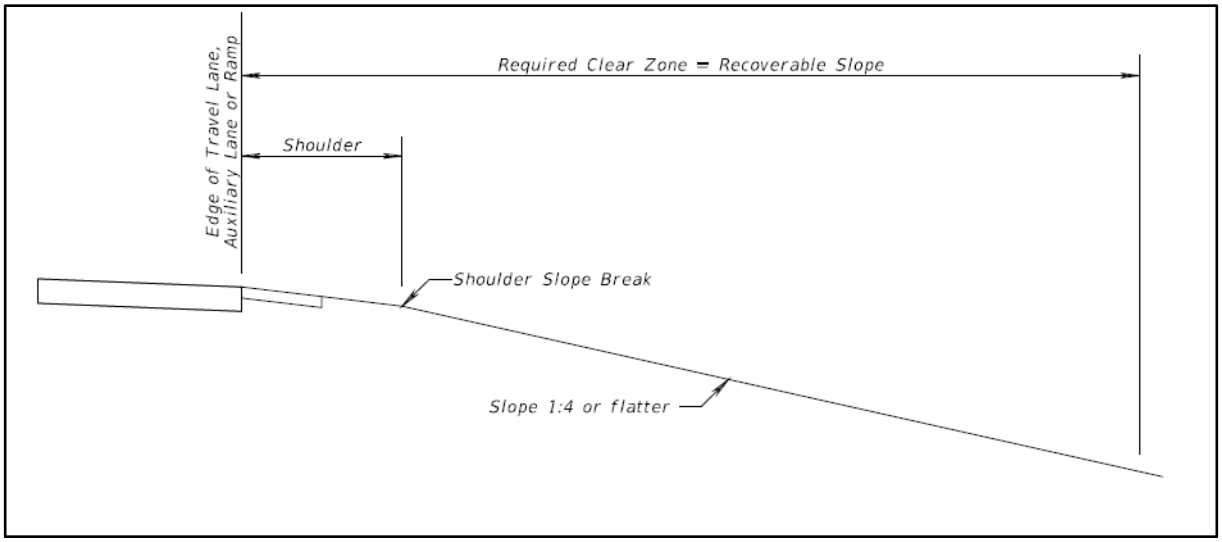
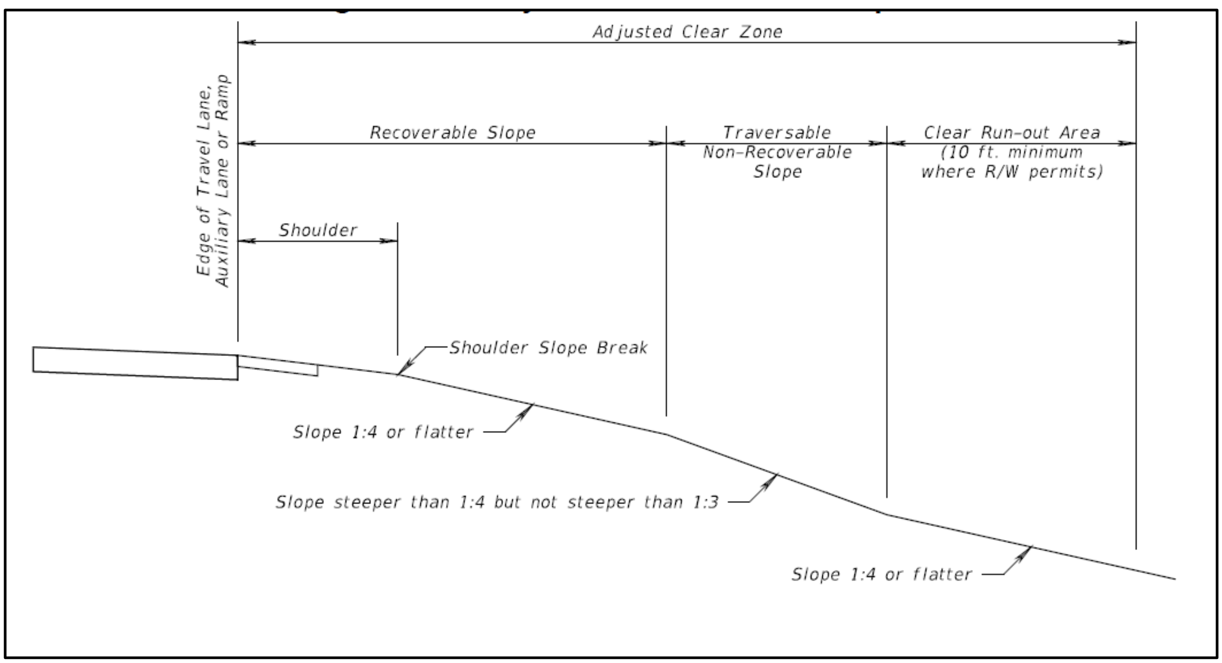
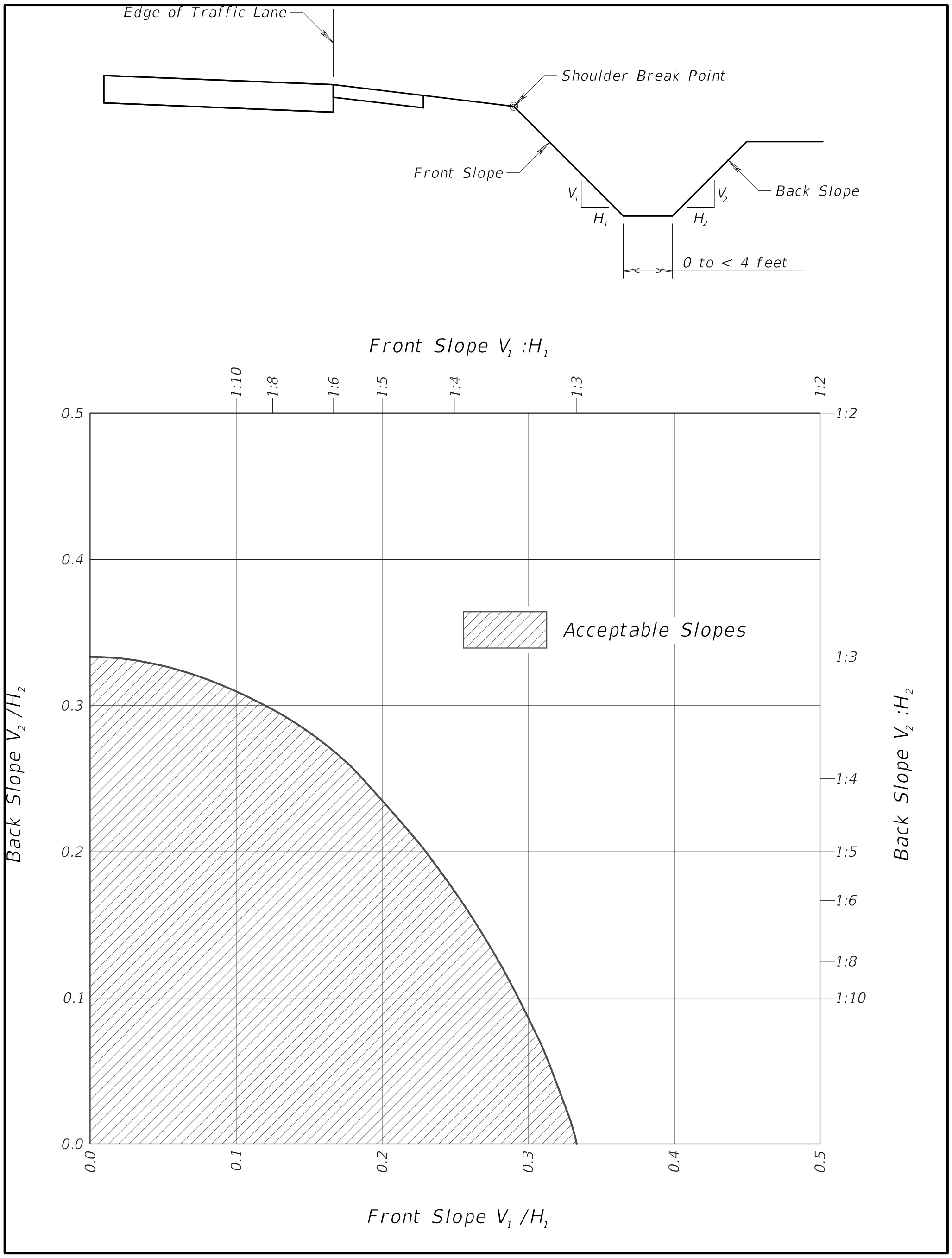


Figure 4 – 3 Adjusted Clear Zone Concept



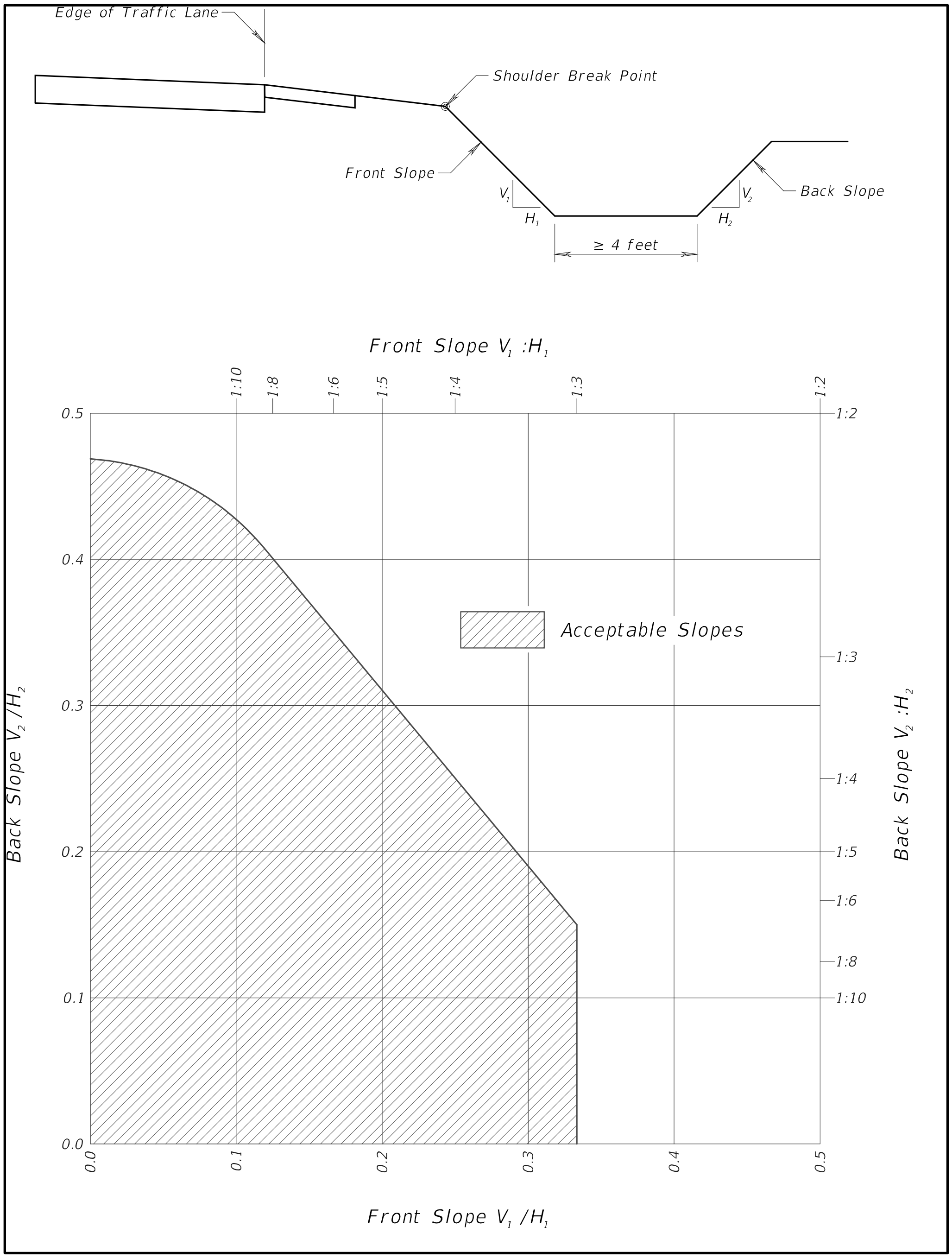
Roadside ditches may be included within the clear zone if properly designed to be traversable. Acceptable cross section slope criteria for roadside ditches within the clear zone is provided in Figure 4 – 4 Roadside Ditches – Bottom Width 0 to < 4 Feet and Figure 4 – 5 Roadside Ditches – Bottom Width ≥ 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 distance from a specified point on the roadway to a roadside hazard. Lateral offset to the roadside hazard is measured as follows:

- Curbed roadways - from face of curb.
- Flush shoulder and high-speed curbed roadways - from outside edge of traveled way.

Lateral offsets apply to all roadways and are determined based on the following:

- Type of facility, i.e., flush shoulder or curbed roadway.
- Design speed.
- Design element.
- Project type, i.e., New Construction, Resurfacing (RRR).

Flush shoulder roadways typically have sufficient right of way to provide the required clear zone widths. Therefore, minimum lateral offset for these roadways is based on maintaining a clear roadside for errant vehicles to recover (i.e., maintaining clear zone width provided in Table 4 – 1 Minimum Width of Clear Zone.

Lateral offsets for curbed roadways should be based on clear zone criteria; however, curbed roadways typically do not have sufficient right of way to provide the required clear zone widths. Therefore, minimum lateral offset on these roadways is based on offset needed for normal operation of the roadway.

At times, it may be necessary to place poles (e.g., signal, light, sign) within the sidewalk. Refer to Chapter 8 – Pedestrian Facilities for minimum unobstructed sidewalk width requirements. Table 4 – 2 Lateral Offset provides minimum lateral offset criteria for roadside features and roadside hazards typically encountered and considered functionally necessary for normal operation of the roadway, e.g., signing, lighting, landscaping, and utilities.

For crashworthy objects, meet or exceed the minimum lateral offset criteria provided in Table 4 – 2 Lateral Offset. Locate objects that are not crashworthy as close to the right of way line as practical and no closer than the minimum lateral offset criteria provided. When a roadside hazard is placed behind a barrier that is justified for other reasons, the minimum lateral offset to the object equals the setback requirements (deflection distance) of the barrier. Additional information on barrier placement and permissible attachments can be found in the ***FDOT Design Manual, Chapter 215.***

~~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. Minimum Width of Clear Zone.~~

~~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 ≤ 25 (mph)	Urban Curbed Roadways Design Speed ≤ 45 (mph)	All Other
Above Ground Objects[1]	1.5 ft. from Face of Curb[3,4]	4 ft. from Face of Curb[2,4]	Clear Zone Width
Drop Off Hazards[5]~~[3]~~	Clear Zone Width	Clear Zone Width	Clear Zone Width
Water Bodies	Clear Zone Width	Clear Zone Width	Clear Zone Width
Canal Hazards	See Section B.2.c	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. For very low-volume roads, ≤ 400 vpd, a minimum of 1.5 feet of clearance is desirable but may be reduced to 6” from the face of curb where the corridor is constrained. ***AASHTO’s Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400), 2001*** provides additional information.

3. May only be used in areas where development patterns and land use would qualify as an Urban Center or Urban Core Context Classification.
   a. Urban Center - Mix of uses set within small blocks with a well-connected roadway network. Typically concentrated around a few blocks and identified as part of the community, town, or city of a civic or economic center.
   b. Urban Core - Areas with the highest densities and with building heights typically greater than four floors. Many are regional centers and destinations. Buildings have mixed uses, are built up to the roadway, and are within a well-connected transportation network.

~~2.~~4. A design variation for failure to meet clear zone criteria is not required for existing, low speed, curbed roadways if the requirements for the placement of above ground fixed objects are met.

~~3.~~5. 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 FDOT's ***Standards Plans***. Drainage features shown in the FDOT's ***Standard Plans*** have the potential for conflict with a motor vehicle or bicyclist either departing the roadway or within a commonly traversed section of a roadway. The FDOT's ~~***The***~~

~~*Department'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 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). Other conditions shall be evaluated using clear zone conditions.

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:

- 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. 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 traveled 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 (Flush Shoulders)

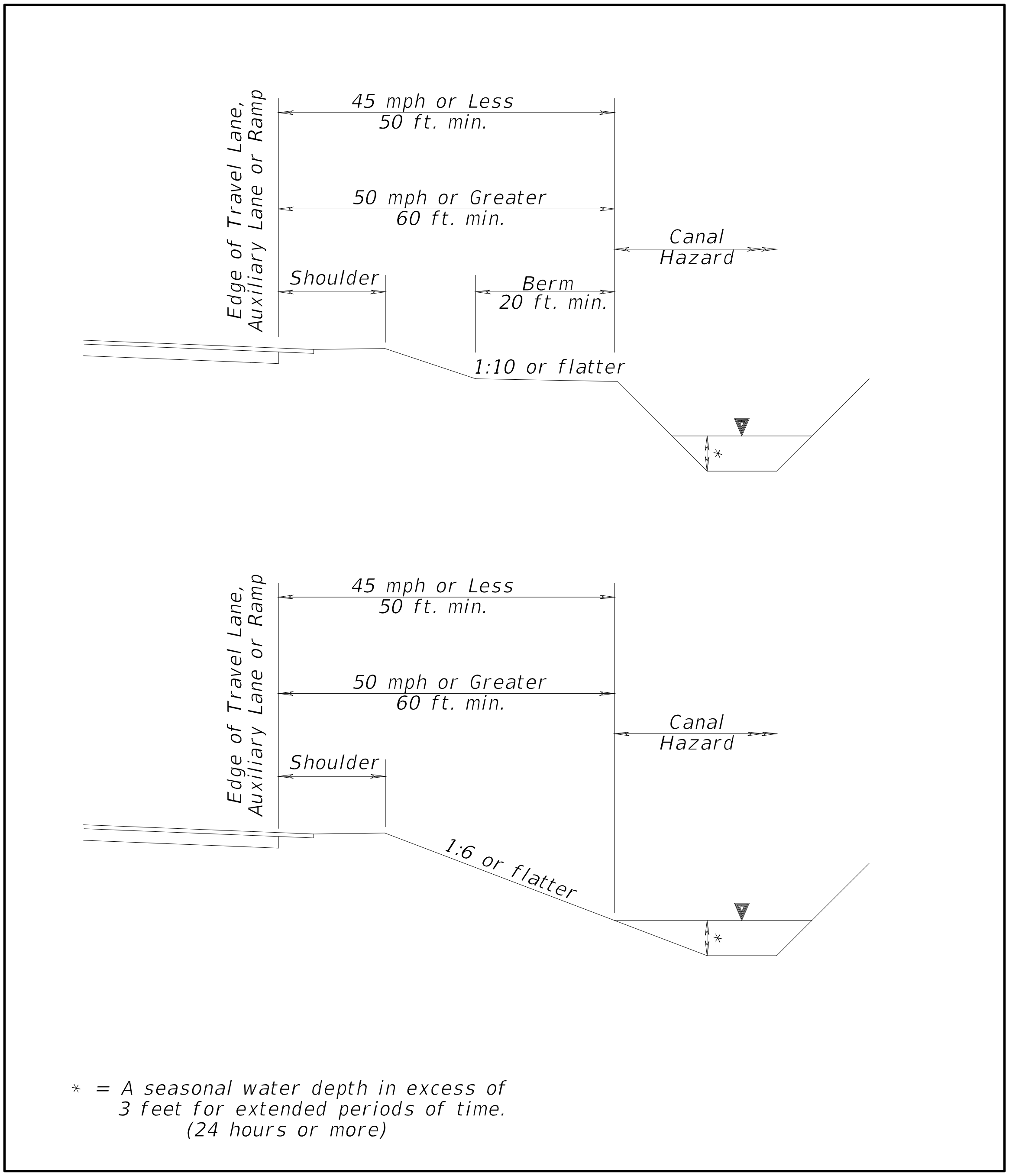
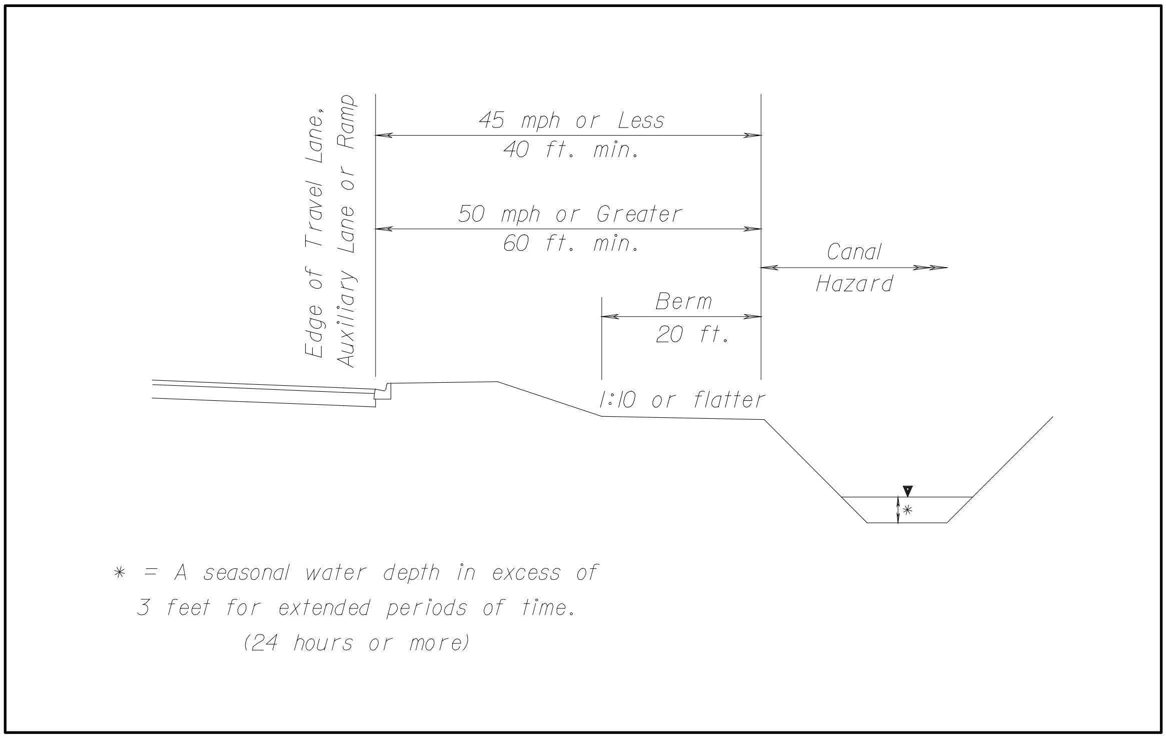


Figure 4 – 7 Minimum Offsets for Canal Hazards (Curbed)



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, bicyclist~~s~~, and pedestrian volumes.
- 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 FDOT's ***Standard Plans*** provide~~s standard d~~ 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 FDOT's ~~***The Department's***~~ ***Drainage Manual*** may be referenced for direction on the use of shoulder gutter.

Curb types such as Type E (height 5" or less with a sloping face equal to or flatter than the Type F~~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 ***FDOT~~Department's~~ Design Manual, Chapter 210 Arterials and Collectors***.
- Directional Median Openings. For examples see the ***FDOT ~~Department's~~ Design Manual, Chapter 212 Intersections.***
- Transit Stops (harmonize with flush shoulder accessible transit stops).

C ROADSIDE SAFETY FEATURES AND CRASH TEST CRITERIA

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, and 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. ***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. Changes have occurred in vehicle design, hardware performance, and testing methodologies, which have led to improvements in crash barrier and roadside design.

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 FDOT~~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. Non-proprietary, standardized devices are detailed in the FDOT's ~~'Departments~~ ***Standard Plans.*** Proprietary products are included on the FDOT's~~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 FDOT ~~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 FDOT's~~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)***, Task Force 13 Guide to Standardized Roadside Safety 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, Approach~~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~~ Approach 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 ***FDOT***~~***The Department's***~~ ***Design Manual, Chapter 215 Roadside Safety*** provides a list of considerations when investigating the need for upgrading barriers and other hardware. The FDOT's ***Standard Plans*** provide standard details for transitioning new barriers to existing barriers. The ***AASHTO Roadside Design Guide*** also provides guidelines for upgrading hardware.

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

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 FDOT's~~The~~ ***Standard Plans*** provide~~s~~ 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 ***Standard Plans*** details should be used as the FDOT 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 FDOT's ~~The Department's~~***Standard Plans*** ~~and~~ 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 FDOT's ***Standard Plans*** 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 FDOT's ***Standard Plans*** provide further information for breakaway lighting supports which are acceptable for use. ~~As a general rule~~Generally, 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 1:6or 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 FDOT's ~~*The Department's*~~ ***Standard Plans*** provide~~s~~ 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 ***AASHTO Roadside Design Guide*** provides additional discussion and guidance on utility poles.

In accordance with Section 337.403, F.S., 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 lateral offset~~clear zone~~ requirements of Table 4 – ~~1~~2 Lateral Offset. 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 and Variations.***

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 traveled 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 curbed streets, the roadside face of the mailbox should be set back from the face of the curb 6 to 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.
- 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 FDOT's ~~***The Department's***~~ ***Standard Plans*** 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.

Additional information on the design and construction of residential and commercial mailboxes, including outdoor cluster boxes can be found on the ***United States Postal Service's Delivery Growth Management*** web page.

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***.

E BARRIERS, APPROACH ~~END~~ TREATMENTS, AND CRASH CUSHIONS

E.1 Roadside Barriers

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.***

E.2 End Treatments

End treatments include trailing~~end~~ anchorages, approach~~end~~ terminals, and crash cushions.~~.~~ ~~Traling~~Trailing~~End~~ anchorages are used to anchor a flexible or semi-rigid barrier to the ground to develop its tensile strength during an impact. Trailing~~End~~ anchorages are not designed to be crashworthy for head~~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. Trailing~~End~~ anchorages are discussed in more detail below.

Approach~~End~~ terminals are basically crashworthy anchorages. Approach terminals are used to anchor a flexible or semi-rigid barrier to the ground at the end of a barrier that is within the minimum clear zone and exposed to approaching traffic. Most approach~~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. Approach~~End~~ terminals are discussed in more detail below.

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, approach~~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 – 23~~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~~several 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~~based on significant crossover crash history.

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

E.7 Temporary Barriers in Work Zones

See Section G Roadside Design in Work Zones for criteria on the use of temporary barriers in work zones.

~~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~~ ~~***DesignStandard 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 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 FDOT's~~Department's~~ ***Standard Plans*** and ***FDOT Design Manual***. Those documents should be referenced for additional details and discussion on the proper use of these systems.

The basis for the FDOT~~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***
- ***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

E.8.a Guardrail

The most commonly used barrier on new construction projects in Florida is the w-beam guardrail system detailed in the FDOT's ~~***the Department's Design***~~ ***Standard Plans***~~***, Index 536-001400***~~ 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 ***FDOT~~Department's~~ 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 FDOT's ***~~Department's~~ Standard Plans*** also provide~~s~~ 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 FDOT's ***~~Department's~~ Standard Plans~~, Index 521-001410~~***. Details are provided for median application, shoulder application and pier protection. Additional information on these barriers is provided in the ***FDOT~~'s~~~~Department's~~ Design Manual, Chapter 215 Roadside Safety***.

The FDOT's ~~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 FDOT~~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 FDOT's~~*The Department's Design*~~ ***Standard Plans****~~, Index 521-002411~~* provides details for crashworthy Pier Protection barriers and the ***FDOT 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 FDOT 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 FDOT~~The Department~~ has used High Tension Cable Barrier (HTCB) in selected locations and continues to install these systems using the FDOT's~~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 FDOT's~~the~~ ***DDS*** website.

It includes the following:

- ***Developmental Standard Plans Instructions D 540-001***
- ***Developmental Standard Plans Index D 540-001***
- ***Developmental Specification, Dev540***

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~~most used temporary barriers in Florida are those adopted for use by the FDOT. The FDOT's~~The department's~~ temporary barriers include:

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

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

Proprietary Temporary Barrier –~~-~~Standards Plans, Index 102-100 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~~ the ***FDOT*** ~~Department's~~ ***Design Manual*** and the Vendor drawings on the FDOT's ***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 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

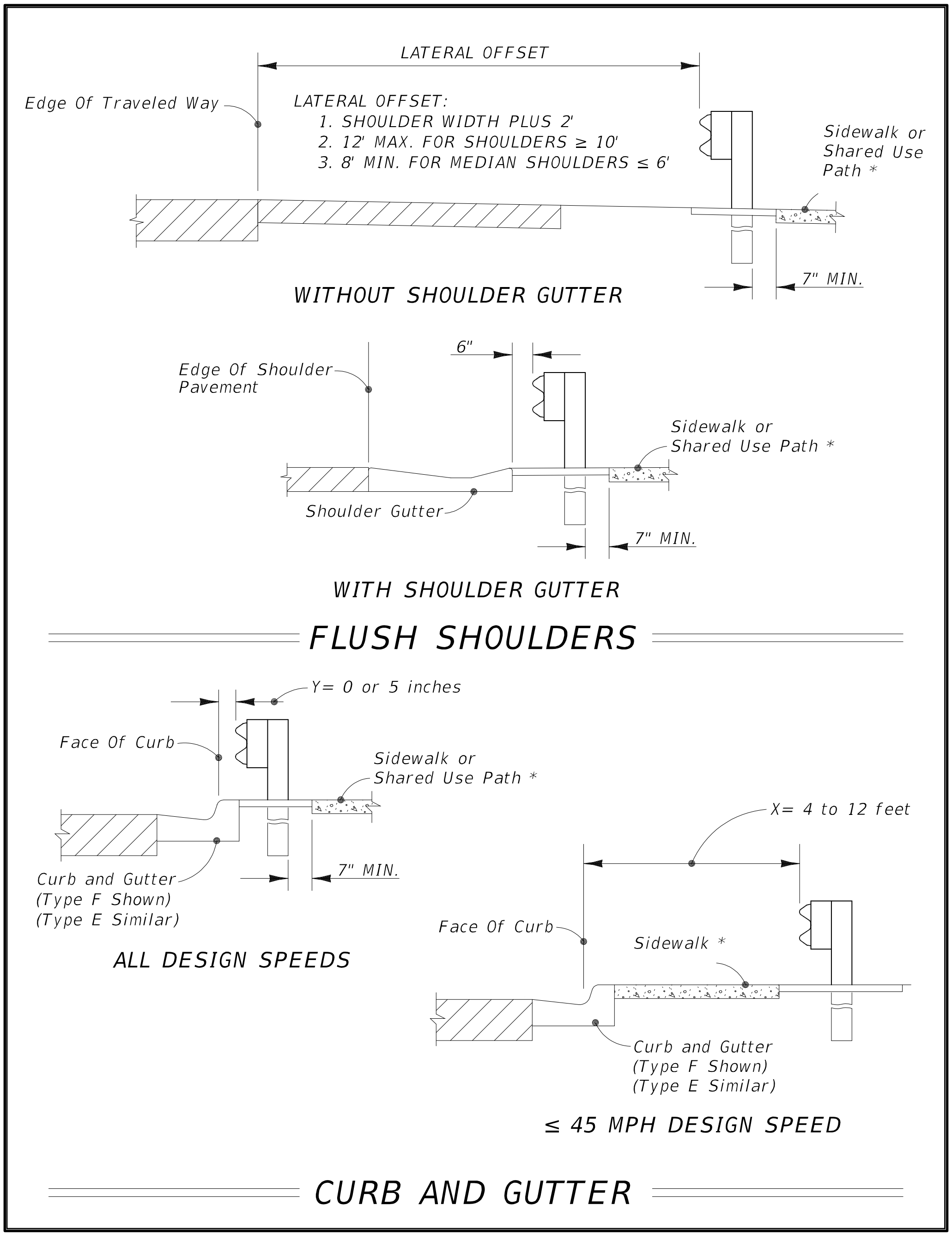
For additional information about considerations for barrier selections refer to the ***AASHTO Roadside Design Guide***. Barrier type selection decisions and warrants should be documented.

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 barriers described above see the ***FDOT*** ~~***Department's***~~ ***Design Manual, Chapter 215 Roadside Safety*** and ***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.~~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***.

These requirements do not apply to devices located within the setback distances detailed in the FDOT's~~the Department's~~ ***Standard Plans*** (e.g., pedestrian/bicycle railing, fencing, noise walls, etc.).

E.8.f.3 Grading

The terrain effects between the traveled way and a barrier can have a significant impact on whether 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.

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.

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 barriers described above, see the ***FDOT~~Department's~~ Design Manual, Chapter 215*** ***Roadside Safety*** 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 FDOT's~~Department's~~ ***Standard Plans*** web page, adjacent to ***Index 536-001*** 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~~panels, and additional posts for increased stiffness. The FDOT's~~The Department's~~ ***Standard Plans*** provide details for several transitions for both permanent and rigid barriers that meet MASH criteria. Additional information on transitions is provided in the ***FDOT'~~s~~ ~~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 ***FDOT~~Department's~~ Design Manual, Chapter 215 Roadside Safety*** for additional information on attachments to barriers.

E.9 End Treatments and Crash Cushions

As previously discussed, end treatments include trailing~~end~~ anchorages, approach~~end~~ terminals, and crash cushions. Details for end treatments for each barrier type described above are detailed in the FDOT's~~Department's~~ ***Standard Plans*** and the ***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~~ G~~g~~uardrail approach end terminals are proprietary devices listed on the ***APL***. MASH compliant a~~A~~pproach 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 A~~anchorages are ~~considered~~non-crashworthy as an approach end treatment, and are not permitted as an approach ~~guardrail~~ end treatment, on the approach end within the Clear Zone, unless shielded by another run of barrier. The FDOT's~~The Department's~~ Type II Trailing ~~End~~ Anchorage, is detailed in the Standard Plans.~~, Index 536-001 .~~

Figures 4-9A and 4-9B below illustrate how to determine when an approach terminal, trailing anchorage or crash cushion should be selected when using guardrail to provide protection for a hazard.

Figure 4 – 9A End Treatment Usage When End of Guardrail is Within Clear Zone of Approaching Near Lane

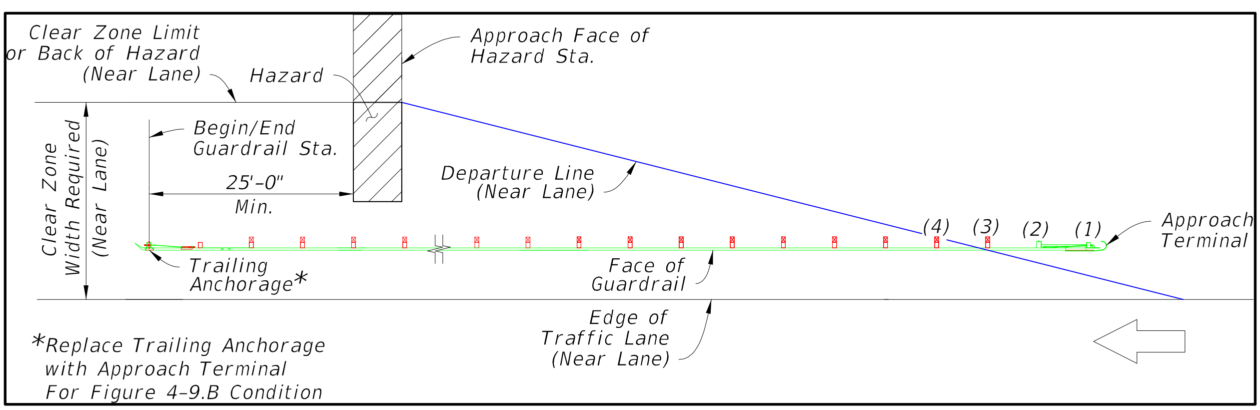
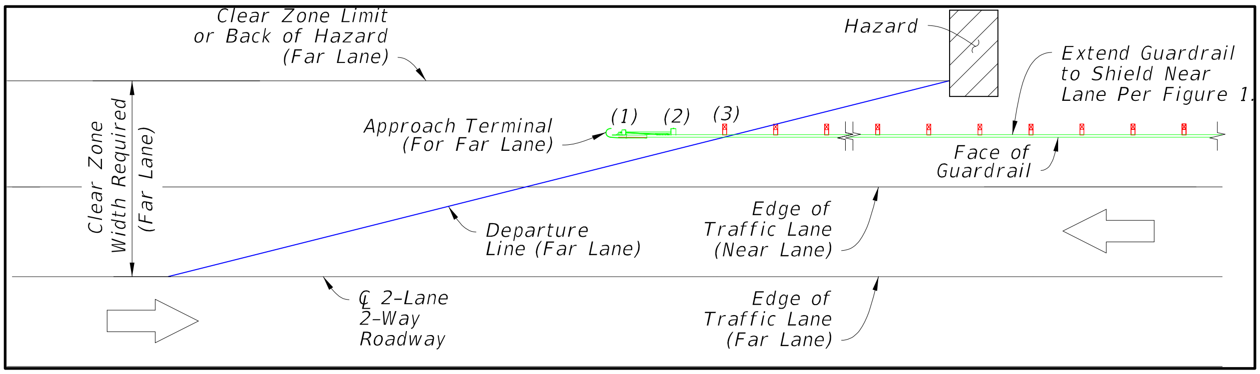


Figure 4 – 9B Approach Terminal Usage When End of Guardrail is Within Clear Zone of Approaching Far Lane (2-Lane, 2-Way Road Shown)



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

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 FDOT's~~Department's~~ ***Standard Plans***. 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 FDOT's~~the Department's~~ developmental design standards discussed above.

E.9.d End Treatments for Temporary Barrier

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

1. Connecting to an existing barrier. Smooth, structural connections are required. Information on connections can be found in the FDOT's~~Department's~~ ***Standard Plans*** and ***APL***.
2. Shield end with a crash cushion as detailed in the FDOT's ***Standard Plans*** ~~***Index 102 Series***~~ 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 FDOT'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 ***FDOT~~Department's~~ Design Manual, Chapter 215 Roadside Safety*** 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 ***FDOT~~Department's~~ Design Manual, Chapter 215 Roadside Safety*** may be referenced for additional information and typical applications.

G ROADSIDE DESIGN IN WORK ZONES

The roadside design concepts presented in the previous sections shall be applied to work zones as appropriate for the type of work being done and to the extent existing roadside conditions allow. This includes providing clear zone and using traffic control devices and safety appurtenances that are crashworthy or properly shielded with crashworthy devices. However, because work zones are temporary and often involve restricted or limited space, modified criteria for clear zones, drop-off conditions and above ground hazards are provided as follows.

G.1 Clear Zone Width in Work Zones

Clear zone is defined in Section B Roadside Topography and Drainage Features. 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.

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	4' Behind Face of Curb	4' Behind Face of Curb
> 45 mph	Same as Flush Shoulder	Same as Flush Shoulder
	Flush Shoulder	
≤ 40	14	10
45 – 50	18	10
55	24	14
60 – 70	30	18
Note: The above clear zone widths apply to medians and roadside conditions other than for roadside canals. Where roadside canals are present, clear zone widths are to conform with the lateral offset distances to canals described in this Chapter.		

The clear zone must be free of above ground fixed objects, water bodies and non-traversable edge drop-offs or critical slopes.

G.2 Above Ground Hazards in Work Zones

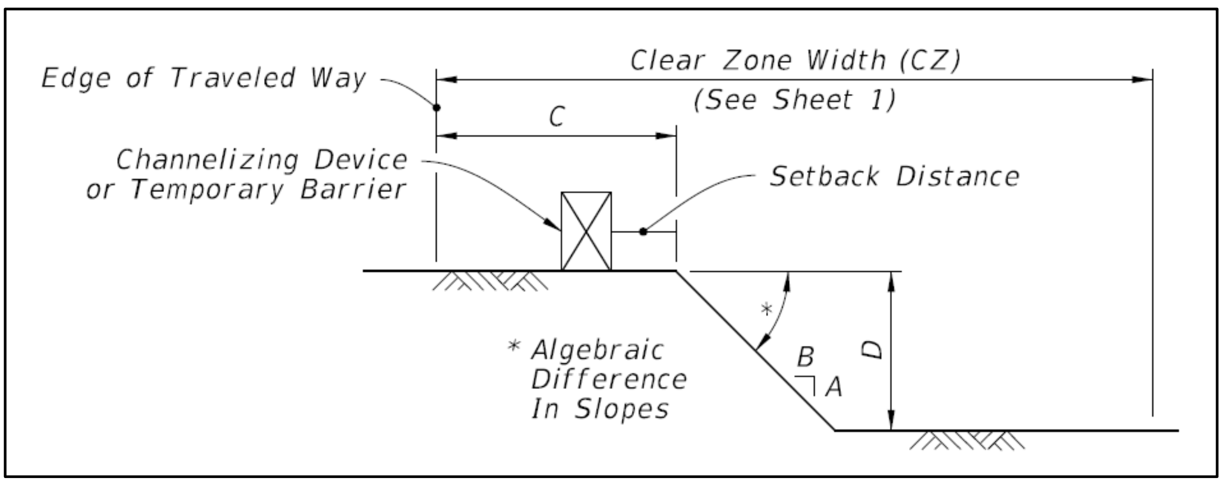
An above ground hazard in work zones is any object, material, or equipment other than temporary traffic control devices that is greater than 4 inches in height, firm and unyielding, and encroaches upon the clear zone. During working hours, above ground hazards in the work zone should be treated with appropriate precautions. During nonworking hours, all objects, materials, and equipment that constitute an above ground hazard must be stored/placed outside of the clear zone or be shielded by a barrier or crash cushion.

G.3 Non-Traversable Edge Drop-Offs, Critical Slopes and Roadside Excavations

Non-traversable edge drop-offs, critical slopes and roadside excavations located within the clear zone are to be addressed as follows:

A drop-off is defined as a drop in elevation, parallel to the adjacent travel lanes, greater than 3" with slope (A:B) steeper than 1:4. In superelevated sections, the algebraic difference in slopes should not exceed 0.25 (See Figure 4 – 10 Drop-off Condition Detail.

Figure 4 – 10 Drop-Off Condition Detail



When an edge drop-off condition occurs within the clear zone, channelizing devices or temporary barriers shall be provided in accordance with Table 4 – 6 Device Requirements for Edge Drop-Offs.

Drop-offs may be mitigated by placing slopes of optional base material. See the ~~the~~ FDOT's ***Standard Specifications, Section 285*** for further information.

Slopes shallower than 1:4 may be required to avoid an algebraic difference in slopes greater than 0.25.

Table 4 – -6 Device Requirements for Edge Drop-Offs

Condition	D (inches)	C (feet)	Device Required
1	>3	2 - 12	Temporary Barrier
2	>3 to ≤5	12 - CZ	Channelizing Device
3	>5	2 - 12	Temporary Barrier
4	Removal of Bridge or Retaining Wall Barrier		Temporary Barrier
5	Removal of portions of Bridge Deck		Temporary Barrier

Footnotes:

1. Do not allow any drop-off conditions greater than 3 inches within two feet of traveled way.
2. For Conditions 1 and 3, channelizing devices and placement of slopes 1:4 or flatter constructed of base material per the ***FDOT Specifications Section 285*** may be used in lieu of temporary barriers. Slopes shallower than 1:4 may be required to avoid algebraic difference in slopes greater than 0.25.
3. For Conditions 1 and 3 any drop-off condition that is created and restored within the same work period will not be subject to the use of temporary barriers. However, channelizing devices will be required.
4. When permanent curb heights are ≥ 6", no channelizing device will be required.

A setback distance appropriate for the type of barrier selected shall be provided. For further information on setback requirements for various types of barriers, see the FDOT's ***Standard Plans.***

Drop-offs adjacent to pedestrian facilities shall be provided with pedestrian longitudinal channelizing devices, temporary barrier wall, or approved handrail. Adjacent to pedestrian facilities, a drop-off is defined as:

a) a drop in elevation greater than 10" that is closer than 2 feet from the edge of the sidewalk or shared use path, or

b) a slope steeper than 1:2 that begins closer than 2 feet from the edge of the sidewalk or shared use path when the total drop-off is greater than 60".

G.4 Temporary Barriers in Work Zones

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 and roadside excavations – see Section G.1.
- Shield above ground hazards, including roadside structures, falsework for bridges, material storage sites and/or other exposed objects.
- Provide positive protection for workers.
- Separate two-way traffic.
- Separate pedestrians from vehicular traffic.

The decision to use temporary barriers for conditions not specifically addressed in Section G.1 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 FDOT's ***Standard Plans, MUTCD*** and the ***AASHTO Roadside Design Guide*** provide additional information and guidance on the use of temporary barriers in work zones.

H~~G~~ REFERENCES FOR INFORMATIONAL PURPOSES

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

- AASHTO Roadside Design Guide
  https://store.transportation.org/Item/CollectionDetail?ID=105/
- Task Force 13 Guide to Standardized Roadside Safety Hardware
  http://www.tf13.org/Guides/
- FHWA Web Site
  http://safety.fhwa.dot.gov/roadway_dept/
- FDOT Design Manual
  http://www.fdot.gov/roadway/FDM/
- FDOT Standard Plans for Road and Bridge Construction (Standard Plans)
  http://www.fdot.gov/design/standardplans/
- *FDOT Structures Design Guidelines*
  http://www.fdot.gov/structures/StructuresManual/CurrentRelease/StructuresManual.shtm
- FDOT Drainage Manual,
  http://www.fdot.gov/roadway/Drainage/ManualsandHandbooks.shtm
- Florida Strategic Highway Safety Plan ~~2016~~
  https://www.fdot.gov/safety/shsp/shsp.shtm

  ~~***http://www.fdot.gov/safety/SHSP2016/SHSP-2012.shtm***~~
  ~~***http://www.fdot.gov/safety/SHSP2012/SHSP-2012.shtm***~~

CHAPTER 5

PAVEMENT DESIGN AND CONSTRUCTION

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CHAPTER 5

PAVEMENT DESIGN AND CONSTRUCTION

A INTRODUCTION

The function of the pavement or roadway surface is to provide a safe and efficient travel path for vehicles using the street or highway. The pavement should provide a good riding surface with a minimum amount of distraction to the driver. The pavement friction characteristics should be such that adequate longitudinal and lateral forces between the vehicle tires and the pavement can be developed to allow a margin of safety for required vehicle maneuvers. These characteristics should be provided at the highest reasonable level for the expected pavement surface, weather conditions, and the anticipated operational characteristics of the facility. Resurfacing of the existing pavement is discussed and included under ***Chapter 10 – Maintenance and Resurfacing*** of this manual.

In order for the pavement to perform its function properly, the following objectives shall be considered in the design and construction of the pavement:

- Provide sufficient pavement structure and the proper pavement material strength to prevent pavement distress prior to the end of the design period.
- Develop and maintain adequate skid resistance qualities to allow for safe execution of braking, cornering, accelerating, and other vehicle maneuvers.
- Provide drainage to promote quick drying and to reduce the likelihood of hydroplaning and splashing.
- ~~Provide a Safety Edge treatment adjacent to the travel lane on roadways without curb or paved shoulders and with posted speed 45 mph or greater.~~

B PAVEMENT DESIGN

B.1 Pavement Type Selection

For new construction and major reconstruction projects, the designer should determine the type of pavement to be constructed utilizing formal analysis of existing and anticipated conditions. High volume roadways where a significant amount of truck traffic (>10%) exists may warrant consideration for special asphalt pavement designs and for rigid pavement designs. The FDOT ~~Department~~ has a documented procedure patterned after the ***~~1993~~ AASHTO Guide for Design of Pavement Structures, Appendix B***. This procedure may be found in the FDOT's ~~Department's~~ ***Pavement Type Selection Manual (20193)***.

B.1.a Unpaved Roadway Material Selection

The material chosen should be locally available when possible. Frequency of grading and replacement of material from loss due to erosion should be evaluated. A life cycle economic analysis should be performed to determine suitable material type. For example: Reclaimed asphalt pavements (RAP) from milling operations provide for a suitable all weather material and can be considered for unpaved roads.

The material chosen should exhibit low potential for losses due to wind, traffic and water erosion. EPA's publication AP-42 contains methodology for estimating the dust generation potential for unpaved road surfaces. Proper gradation of the chosen material is critical for its success. Designers should consider flexible or rigid pavements where runoff from unpaved roads may impact surface waters.

Designers may consult with AASHTO's ***Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400), 2001*** and ***FHWA's Gravel Roads Construction and Maintenance Guide, August 2015*** for further guidance regarding material selection.

B.2 Structural Design

The pavement shall be designed and constructed so the required surface texture is maintained and its structure retains an adequate level of serviceability for the design period. The strength of the pavement materials shall be sufficient to maintain the desired roadway cross section without the formation of ruts or other depressions which would impede drainage. Subgrade strength and subgrade drainage are major factors to be considered in pavement design. Where high ground water conditions are present, adequate clearance to the bottom of the pavement base is necessary for good pavement performance and to achieve the required compaction and stability during construction operations.

The FDOT's pavement design manuals, including the ***Flexible Pavement Design Manual, 2021*** and ***Rigid Pavement Design Manual, 2021***, are recommended as a guide for both flexible and rigid pavement design. Other design procedures are available including the ***AASHTO Guide for Design of Pavement Structures***,~~, 1993;~~ and procedures which have been developed by the Portland Cement Association, the American Concrete Pavement Association, and the Asphalt Institute. The selection of the design procedure and the development of the design data must be managed by professional personnel competent to make these evaluations.

B.3 Skid Resistance

Pavements shall be designed and constructed to maintain adequate skid resistance for as long a period as the available materials, technology, and economic restraints will permit, thus eliminating cost and hazardous maintenance operations.

The results of relevant experience and testing (i.e., tests conducted by the ~~the~~ FDOT~~Department~~'s Materials Office) should be used in the selection of aggregate and other materials, the pavement mix design, the method of placement, and the techniques used for finishing the pavement surface. The design mixes should be monitored by continuous field testing during construction. Changes to the design mix or construction procedures must be made by qualified pavement designers and laboratory personnel ONLY.

The use of transverse grooving in concrete pavements frequently improves the wet weather skid resistance and decreases the likelihood of hydroplaning. This technique should be considered for locations requiring frequent vehicle maneuvers (curves, intersections, etc.) or where heavy traffic volumes or high speeds will be encountered. The depth, width, and spacing of the grooves should be such that control of the vehicle is not hindered.

B.4 Drainage

Adequate drainage of the roadway and shoulder surfaces should be provided. Factors involved in the general pavement drainage pattern include pavement longitudinal and cross slopes, shoulder slopes and surface texture, curb placement, and the location and design of collection structures. The selection of pavement cross slopes should receive particular attention to achieve the proper balance between drainage requirements and vehicle operating requirements. The use of curbs or other drainage controls adjacent to the roadway surface should be avoided, particularly on high-speed facilities. Specific requirements for cross slopes and curb placement are given in ***Chapter 3 – Geometric Design***.

B.4.a Unpaved Roadway Drainage

Properly graded unpaved roadways require less maintenance and suffer less material loss. Designers should strive to provide adequate cross slope, shoulder and swale profiles wherever possible. Typical cross slopes should be 2% with 1.5% minimum. During maintenance grading, the operator should ensure that the final shoulder does not become higher than the travel lane edge to prevent ponding of water on the roadway.

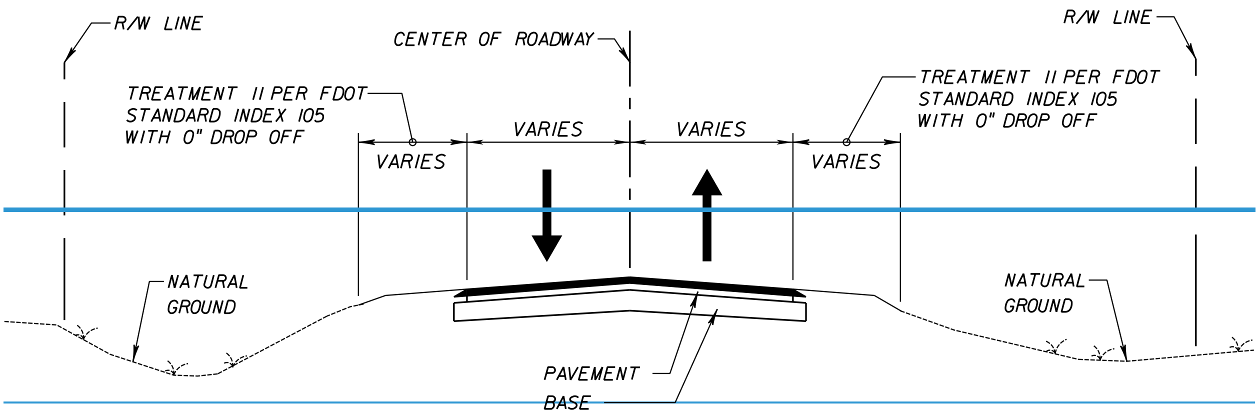
Designers may consult with AASHTO's publication ***Guidelines for Geometric Design of Very Low-Volume Roads (ADT < 400), 2001*** and FHWA's ***Gravel Roads Construction & Maintenance Guide, August 2015*** for further guidance regarding proper cross slope profiles for unpaved roads.

B.5 Shoulder Treatment

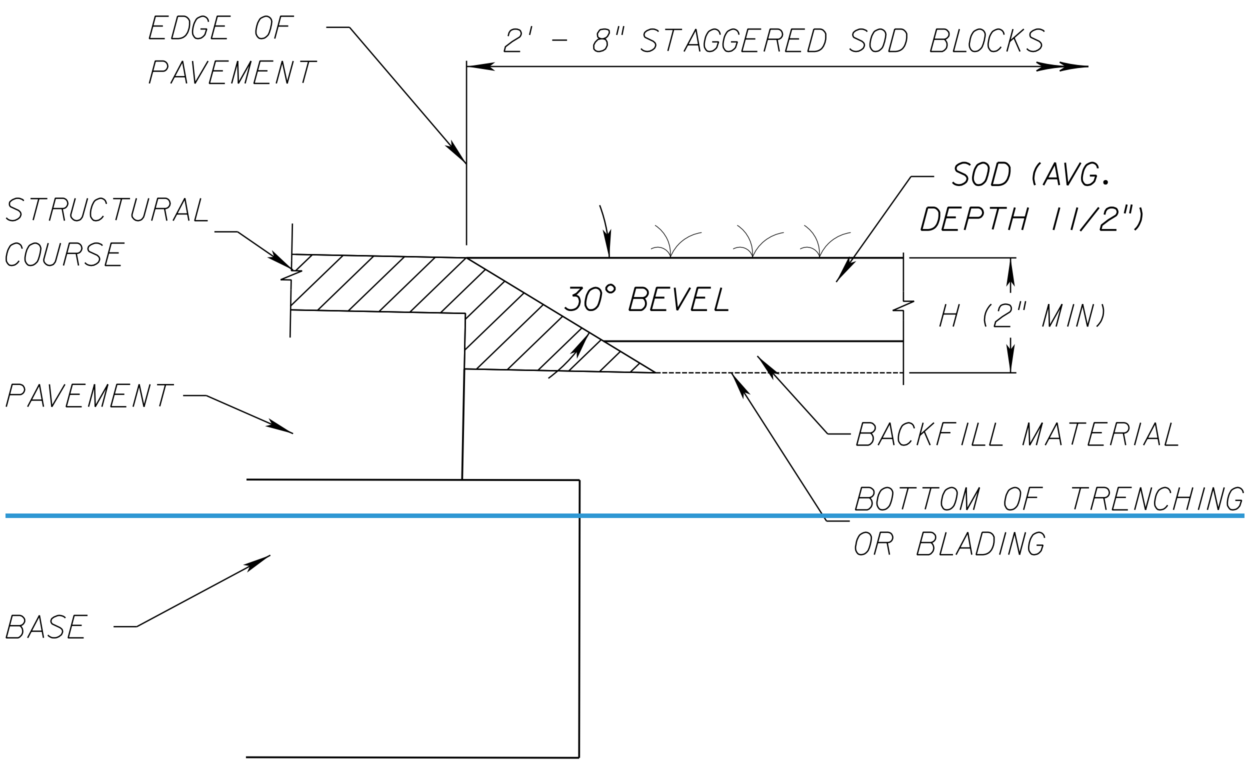
The primary function of the shoulder is to provide an alternate travel path for vehicles in an emergency situation. Shoulders should be capable of providing a safe path for vehicles traveling at roadway speed, and should be designed and constructed to provide a firm and uniform surface capable of supporting vehicles in distress. Particular attention shall be given to provide a smooth transition from pavement to shoulder. Shoulder pavement may be provided to improve drainage of the roadway, provide lateral support of roadway pavement, to serve bicyclists~~bicycles~~, pedestrians, and transit users, and to minimize shoulder maintenance. See ***Chapter 3 – Geometric Design*** for additional information and criteria for shoulders.

Safety Edge is a technology that mitigates vertical drop offs. The Safety Edge provides a higher probability of a vehicle returning safely to the travel lane when it drifts off the pavement. ~~The wedge shape eliminates tire scrubbing and improves vehicle stability as it crosses a drop-off. Details of the Safety Edge are included in Figures 5 – 1 Two Lane road with Safety Edge and 5 – 2 Safety Edge detail (no Paved Shoulders).For further information on Safety Edge,~~ See ***Chapter 10 – Maintenance and Resurfacing, Section C.3.a Pavement Safety Edge*** for additional information and requirements for Safety Edge.

~~Figure 5 – 1~~
~~Two Lane Road with Safety Edge~~



~~Figure 5 – 2~~
~~Safety Edge Detail (No Paved Shoulders)~~



FOR 2" ≤ H ≤ 5"

SINGLE LIFT
~~SAFETY EDGE DETAIL~~

~~Shoulder pavement may be provided to improve drainage of the roadway, to serve bicycles, pedestrians and transit users, and to minimize shoulder maintenance.~~

C PAVEMENT CONSTRUCTION

A regular program of inspection and evaluation should be conducted to ensure the pavement criteria are satisfied during the construction process. Any regular inspection program should include the following:

- The use of standard test procedures, such as AASHTO and the American Society for Testing and Materials (ASTM).
- The use of qualified personnel to perform testing and inspection.
- The use of an independent assurance procedure to validate the program.

~~After construction, the pavement surface shall be inspected to determine the required surface texture was achieved and the surface has the specified slopes. Spot checking skid resistance by approved methods should be considered. Periodic reinspection should be undertaken in conformance with the guidelines described in Chapter 10 – Maintenance and Resurfacing.~~
After construction, the pavement surface shall be inspected to determine the required surface texture was achieved and the surface has the specified slopes. Spot checking skid resistance by approved methods should be considered. Periodic reinspection should be undertaken in conformance with the guidelines described in ***Chapter 10 – Maintenance and Resurfacing***.

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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 nighttime 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 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 may be 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 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 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 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 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.
- 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, 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

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 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 – 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 efficiency of over 100 lumens per watt.

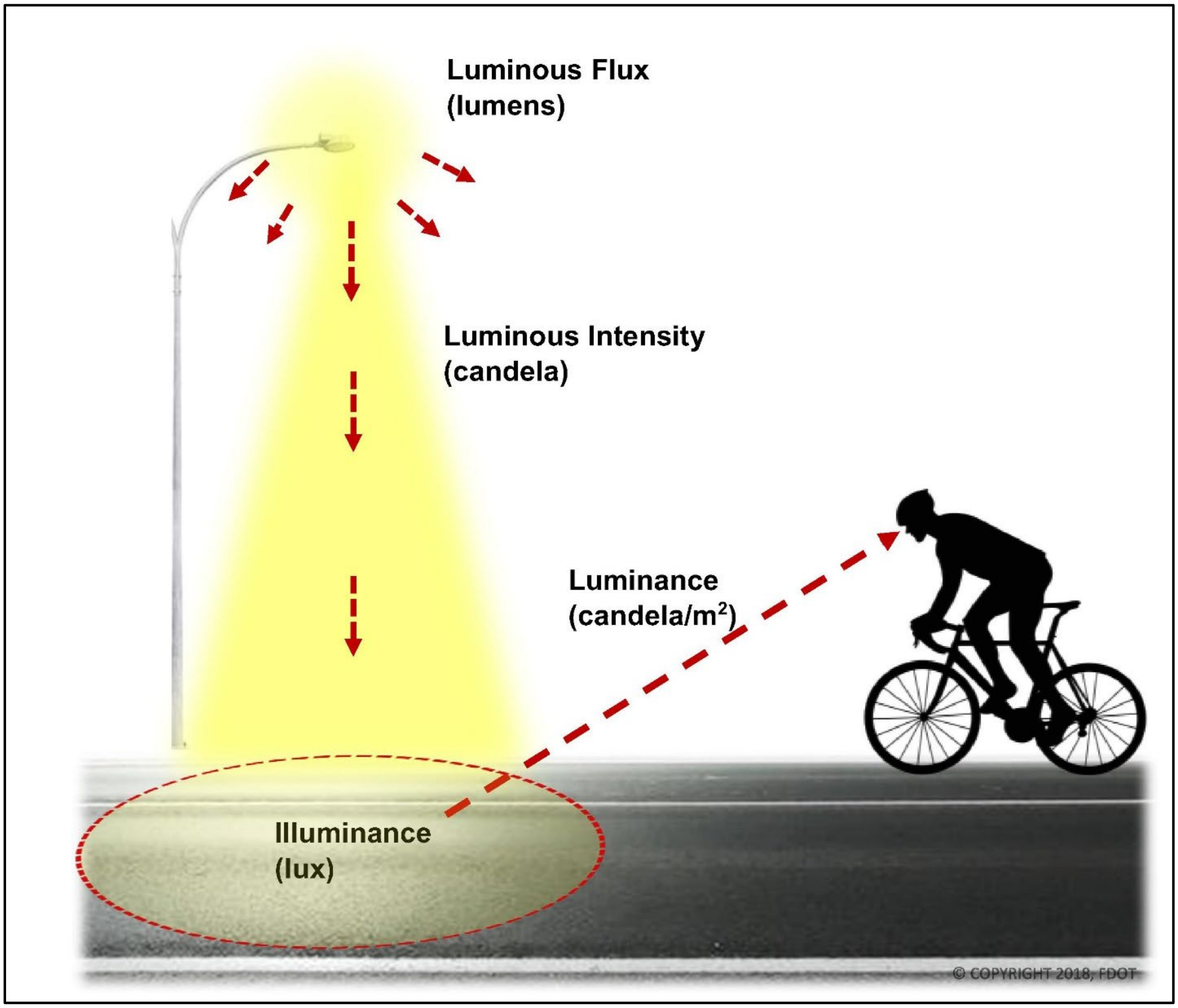
- Metal Halide (MH) Lamps – is used for overhead lighting of commercial parking lots, sports facilities, retail stores and street lighting. Light produced by MH lamps has a CCT of 3800°K to 4000°K which is a white color. The average rated life of a MH lamp can vary from 9,000 to 20,000 hours. MH lamps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

E LIGHTING DESIGN TECHNIQUES

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. 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. The luminance method can be used for straight roadways and streets, based upon the appropriate choice of surface type.

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

The illuminance method determines the amount of light falling on the roadway surface or on vertical surfaces from the roadway lighting system. 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 luminance method 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. 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.***

Table 6 – 1 Road Surface Classifications

Class	Q0*	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. 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 Illuminance and Luminance Design Values

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 next page

Table 6 – 2
Illuminance and Luminance Design Values
(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.

F UNIFORMITY OF ILLUMINATION

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

G.2 Night Lighting

The nighttime illumination level in the underpass of the roadway should be maintained near the nighttime 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. Due to relatively low luminaire mounting heights in underpasses, care should be exercised to avoid glare.

H DECORATIVE ROADWAY LIGHTING

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

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.

J WILDLIFE-SENSITIVE LIGHTING

The lighting on some coastal roadways may affect wildlife, including sea turtles, and may require lower lighting levels, adjusting direction of luminaires, and different types and colors of lighting installations than what might normally be provided. Sea turtles and their habitat (nesting beaches) are afforded protection in accordance with ***Florida's Marine Turtle Protection Act (379.2431, F.S.)*** which restricts the take, possession, disturbance, mutilation, destruction, selling, transference, molestation, and harassment of marine turtles, nests or eggs.

The state of Florida developed the ***Model Lighting Ordinance for Marine Turtle Protection Rule (62B-55, F.A.C.)*** to guide local governments in creating lighting ordinances. Counties and municipalities in Florida that have passed ordinances prohibiting light from reaching the beach can be found on the Municipal Code Corporation web site. Coordinate with the local agencies in proximity to the project for additional requirements and guidance on providing permanent lighting or lighting in work zones.

Wildlife areas of concern can be determined by contacting the Florida Fish and Wildlife Conservation Commission (FWC) at ***MarineTurtle@MyFWC.com***. KMZ layers and Shape files illustrating areas where wildlife sensitive areas occur can be found on the ***FDOT's Office of Environmental Management "OEM Resources"*** web page, under Turtle Lighting.

An interactive map of wildlife sensitive areas can also be found in the ***Florida Geographic Data Library (FGDL),*** and will show areas of the state where wildlife sensitive lighting measures should be implemented. Use the key word "turtle" in the search function. Direct links for download from the ***FGDL*** layers are:

- ***https://download.fgdl.org/pub/state/trtl_sen_light_jul20.zip***
- ***https://download.fgdl.org/pub/state/trtl_drksky_light_jul20.zip***

Additional information can be found on ***FWC's Sea Turtle Lighting Guidelines*** website.

For conventional lighting near a wildlife area of concern, incorporate the following design requirements:

1. Where feasible, orient luminaires away from the wildlife area of concern.
2. Design lighting system using luminaires that meet the following requirements:
   a. The light source for the luminaires must be true red, orange, or amber light-emitting diodes (LEDs) with no more than 1.75% of the spectral power distribution below 560 nm.
   b. The optics must have an IP 66 rating.
   c. The luminaire mounting assembly must be a slip fitter type designed to accommodate a nominal 2 inch pipe size (2-3/8 inch O.D.) arm or a pole top mounting assembly designed to accommodate a 2-3/8 inch pole top tenon.
   d. Luminaires must have a IESNA light distribution curve (IES LM-79) designated by an EPA-recognized laboratory.
   e. Luminaires must meet a minimum pole spacing of 50 feet.

Further information on luminaires which meet the criteria for wildlife sensitive lighting may be found on the FDOT's ***Approved Product List (APL)*** in the Wildlife-Sensitive Conventional Lighting category or FWC's ***Certified Wildlife Lighting Guidelines.*** The AGi32 lighting optimization tool, used in accordance with the settings shown in the FDOT ***Standard Specifications for Road and Bridge Construction, 992-2.4 Luminaires for Wildlife Sensitive Lighting***, may be used to design appropriately spaced lighting.

~~Section~~ J.1 Work Zones in Wildlife Sensitive Areas

For night work along coastal roadways where sea turtles may be affected, incorporate the following for temporary lighting of work zone operations:

1. Direct all work zone lighting away from the beach to avoid illumination of or direct visibility from the beach.

2. Shield luminaires to avoid lighting areas outside of the immediate construction area.

K 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.

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

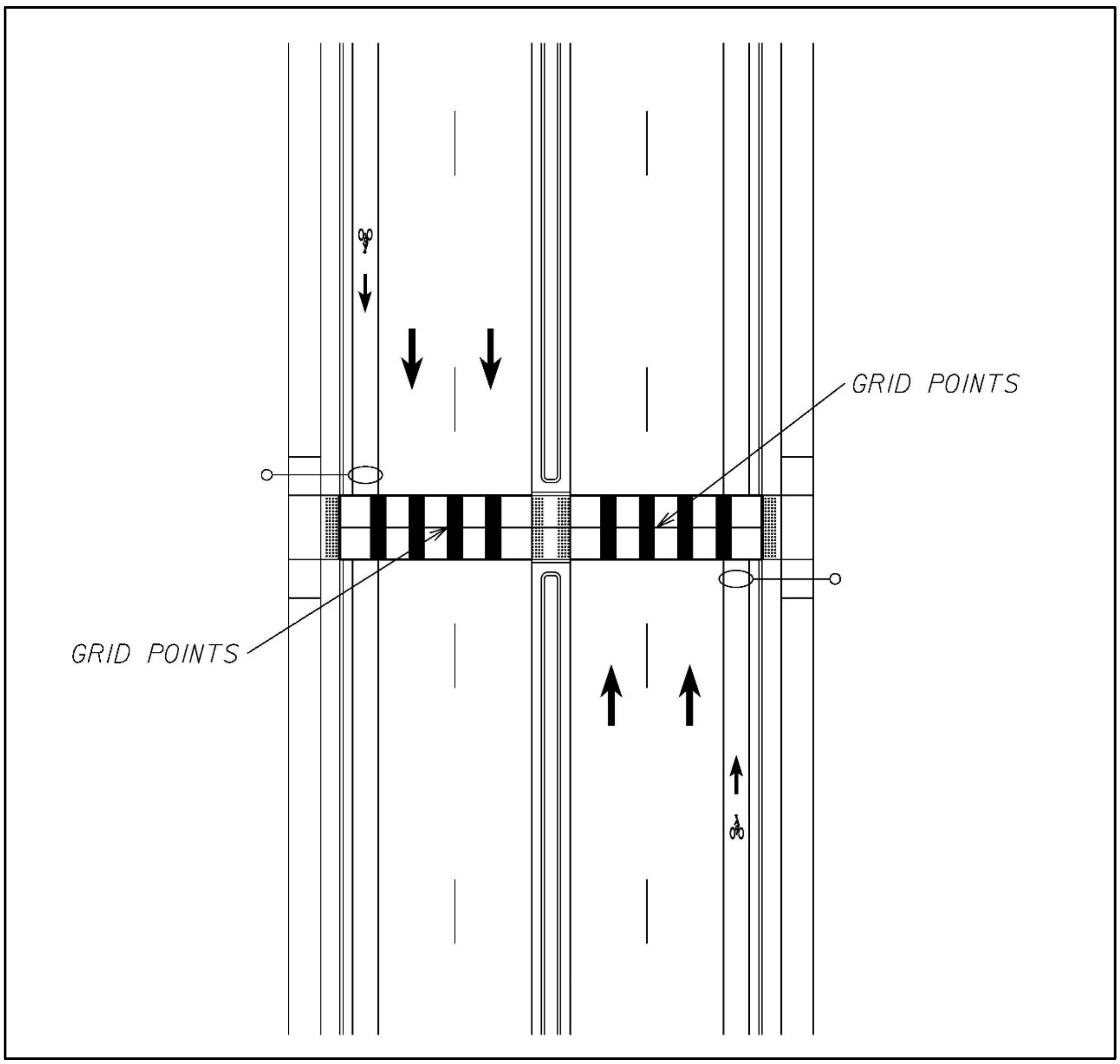
L ROUNDABOUTS

Roundabouts should be supplemented with roadway lighting. Where pedestrians are expected, provide additional lighting of 2.0-foot candles of maintained vertical illumination, measured at 5 feet from the road surface. Calculate the vertical illuminance for the crosswalk on each near side approach entering and exiting the roundabout.

M MIDBLOCK CROSSWALKS

At midblock pedestrian crossings, provide 2.0-foot candles of maintained vertical illumination, measured at 5 feet from the road surface. Calculate the vertical illuminance for the crosswalk on each near side approach.

Figure 6 – 2 Horizontal and Vertical Illuminance for Mid-Block Crosswalk



N 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***.

O 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 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***.

P REFERENCES FOR INFORMATIONAL PURPOSES

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

- Roadway Lighting, ANSI/RP-8-21
  https://blog.ansi.org/ansi-ies-rp-8-21-design-roadway-lighting/#gref

- 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***~~***https://highways.dot.gov/safety/other/visibility/roadway-lighting-resources***

- 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 FDOT~~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~~like 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

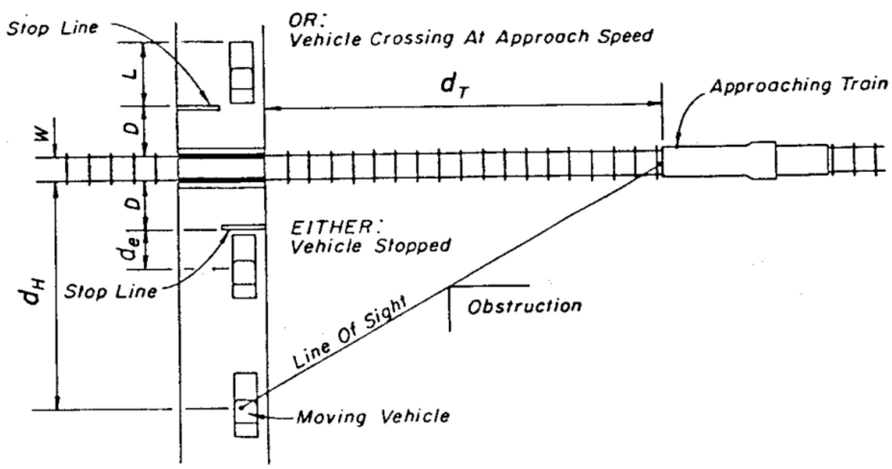
The sight distance required for a vehicle to cross a railroad from a stop is essentially the same as that required to cross a highway intersection as given in ***Chapter 3 – Geometric Design***.

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.

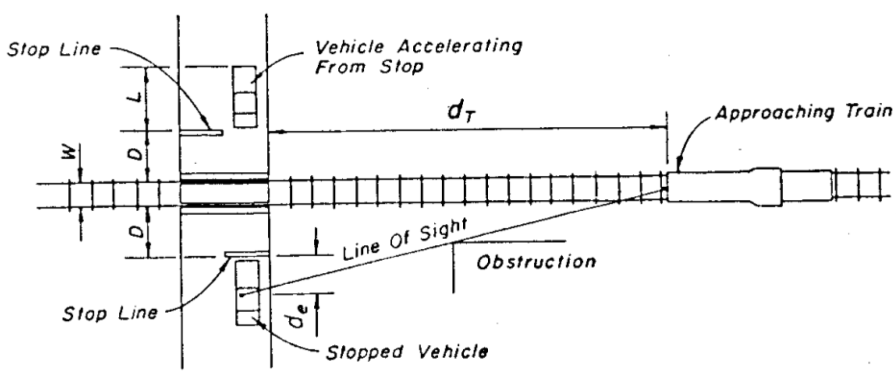
The setback for determining the required clear area for sight distance should be at least 10 feet more than the vehicle offset. Care should be exercised to ensure signal supports and other structures at the crossing do not block the view of drivers preparing to cross the tracks.

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 (Cont')

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:

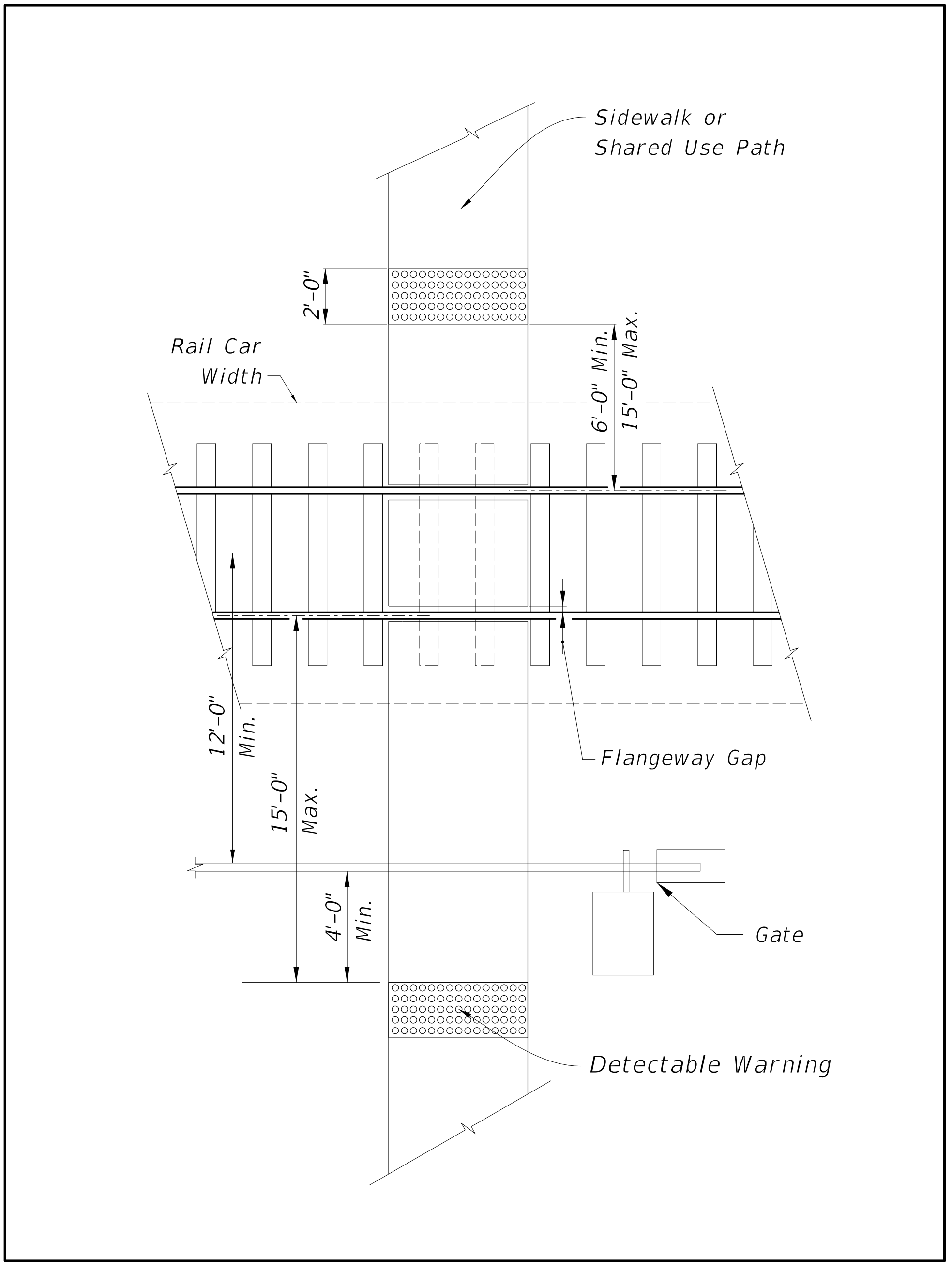
- firm, stable and slip resistant,
- level and flush with the top of rail at the outer edges of the rails, and
- area between the rails align with the top of rail.

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

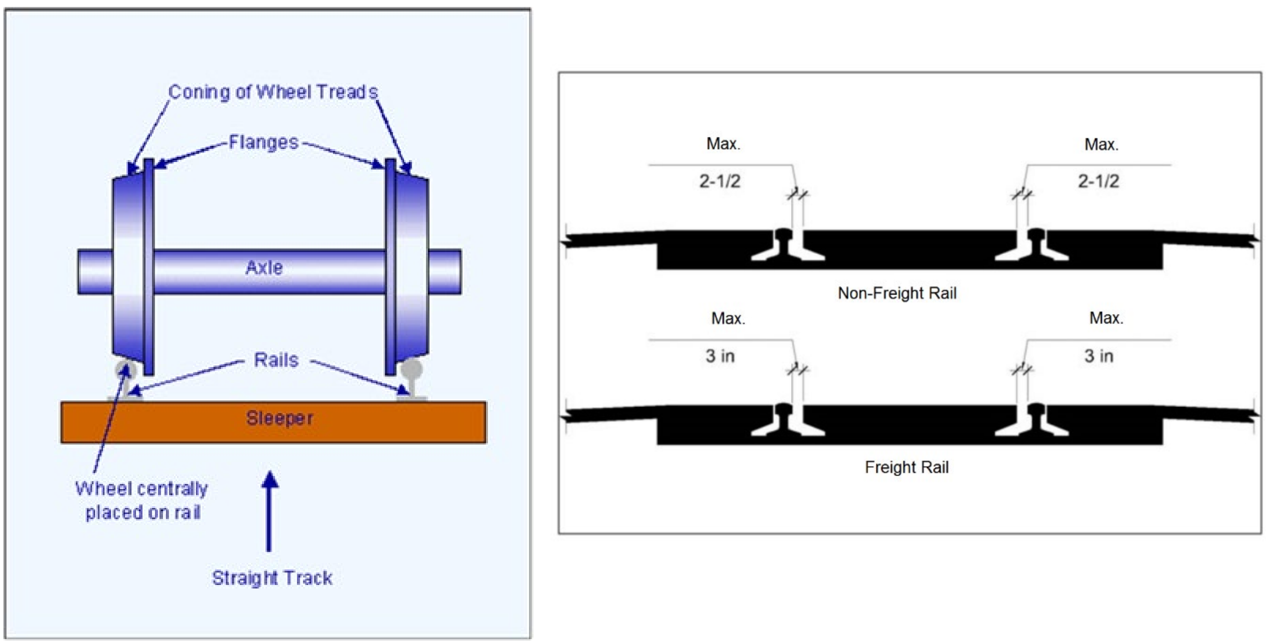


Note: Pedestrian gates may be installed on the outside of the sidewalk/shared use path or in the utility strip.

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 ***2020~~17~~ Florida Building Code, Accessibility 7th Edition ~~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~~always stop. 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~~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"

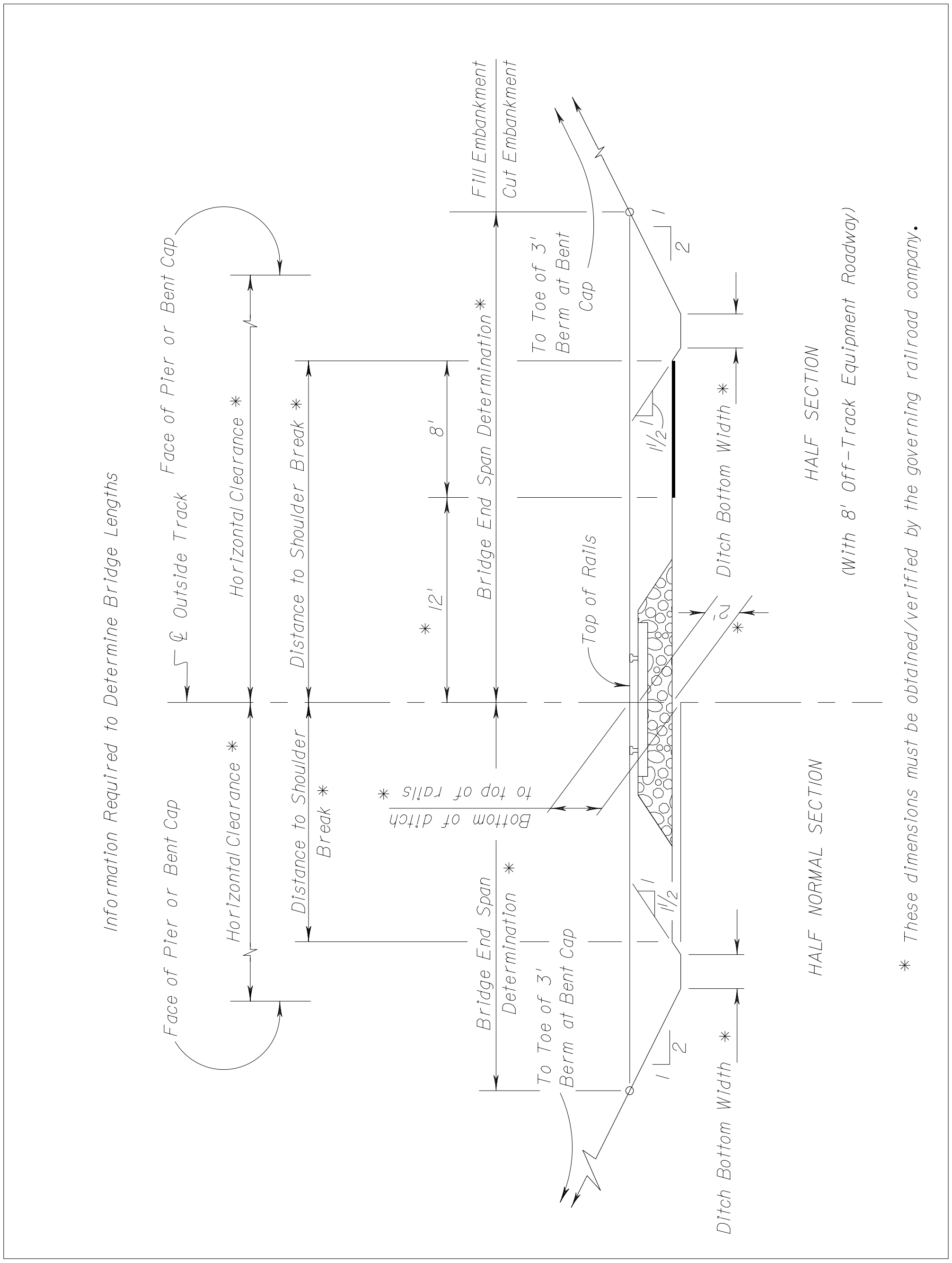
1. Over High Speed Rail Systems, see the latest version of ***American Railway Engineering and Maintenance-of-Way Association (AREMA)*** guidelines, or the design office of the high-speed rail line of interest for specific guidelines and specifications. Over Electrified Railroad, the minimum vertical 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 ***FDOT 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: 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 FDOT ***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.

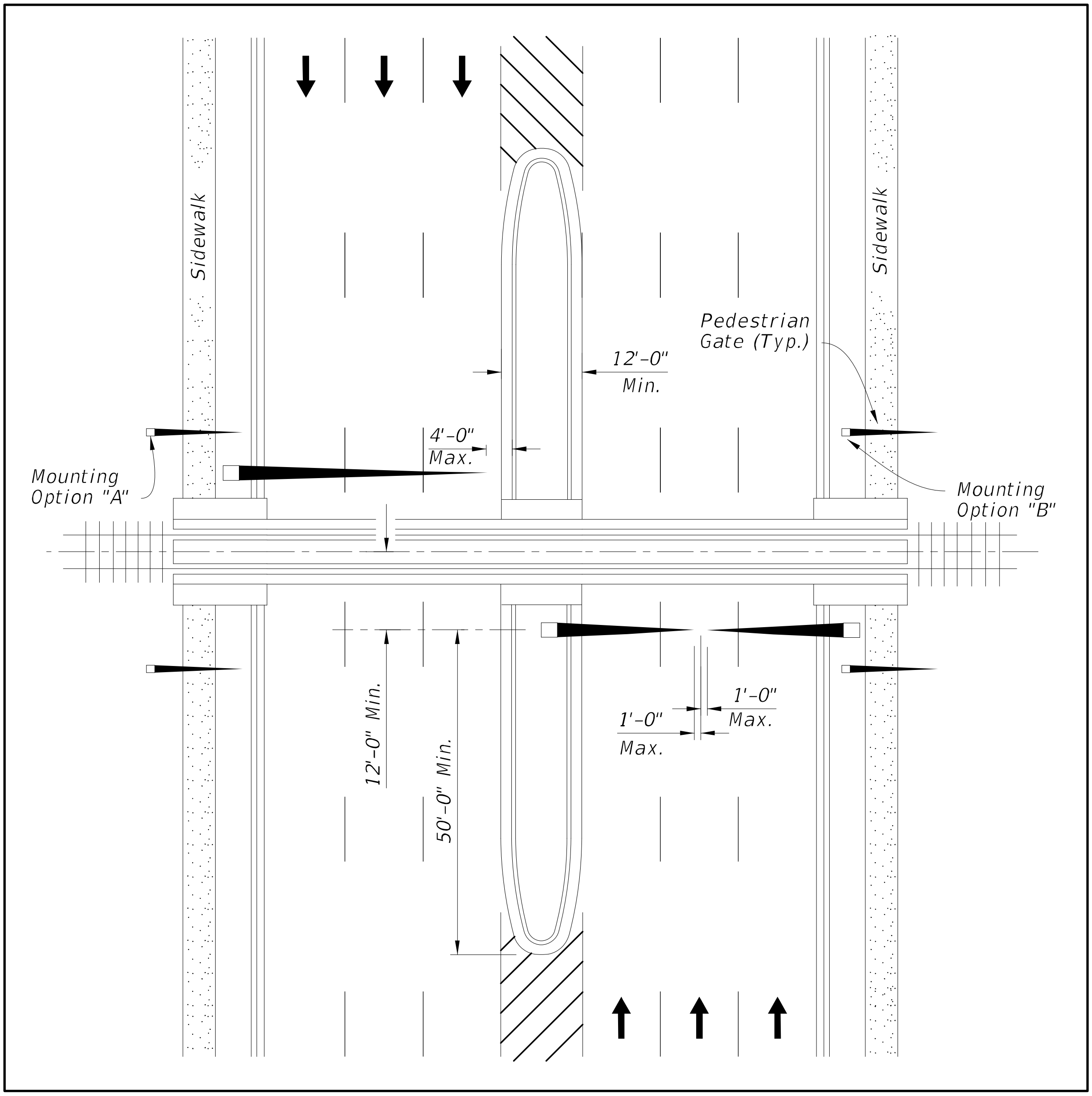
D.9 Traffic Control Devices

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.

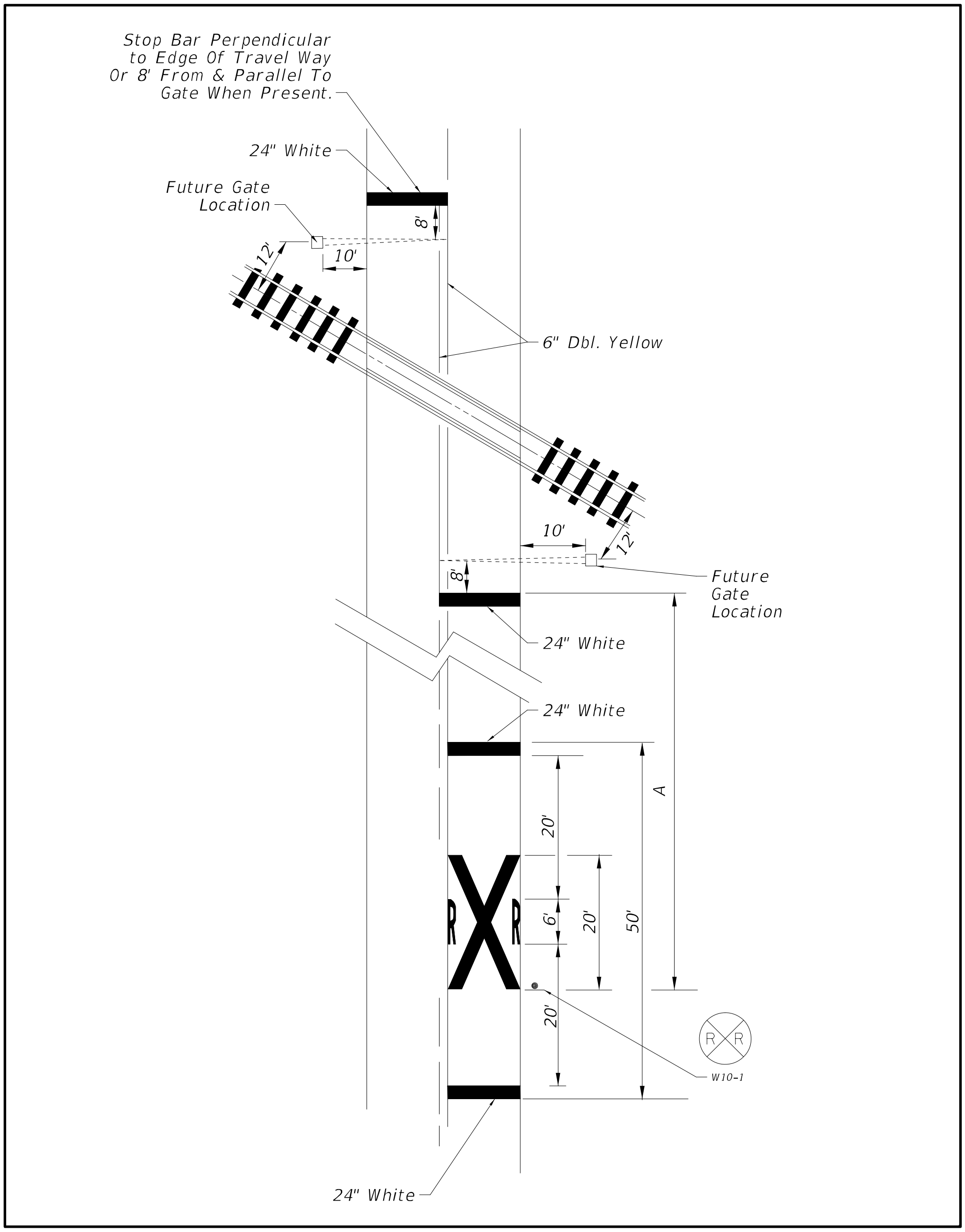
D.12 Crossing Configuration

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 FDOT's Standard Plans.

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**)*

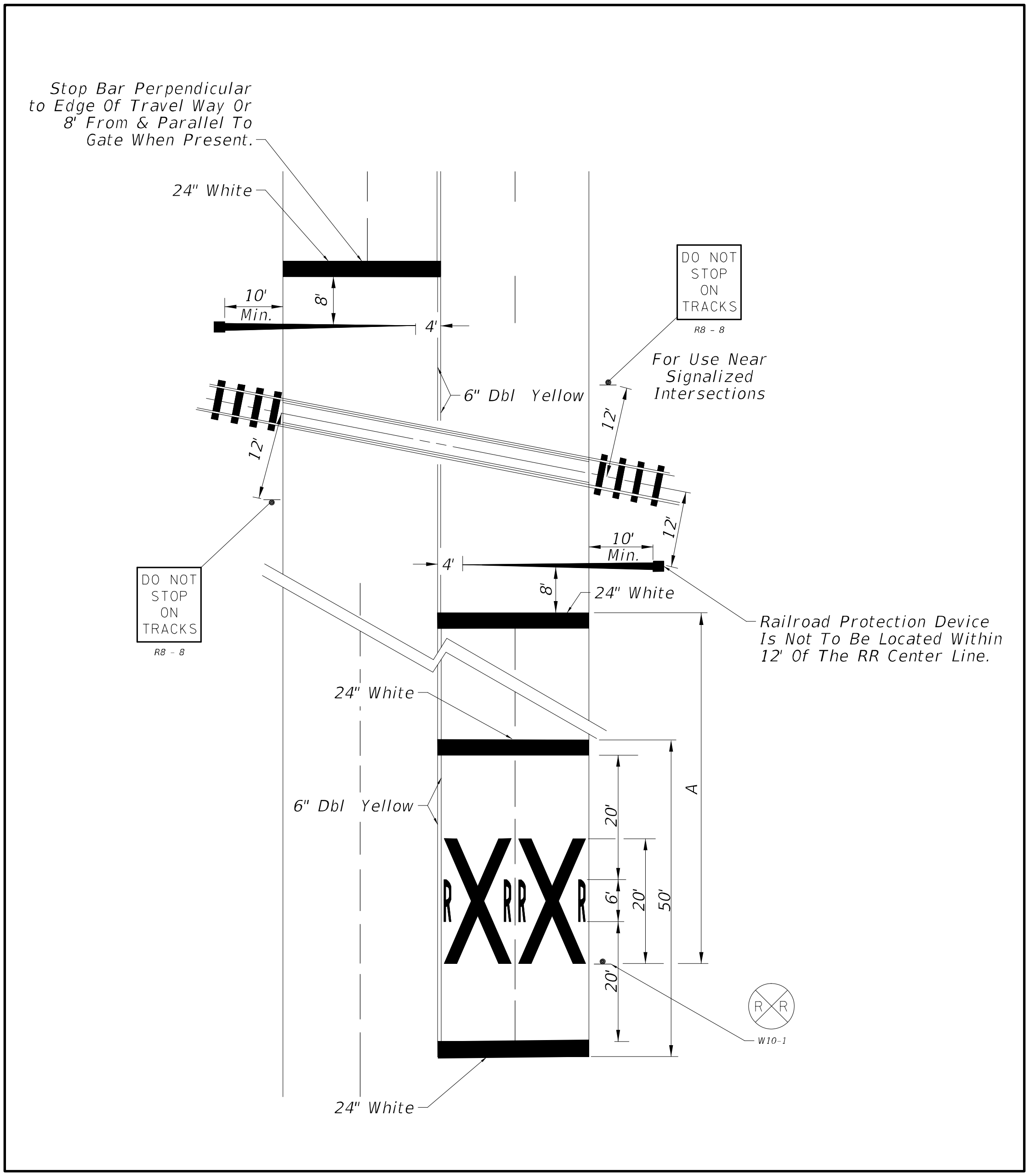
Active rail-highway grade crossings include traffic control devices that give advance notice of the approach of a train. *(Source: **FHWA Railroad-Highway Grade Crossing Handbook**).*

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***.

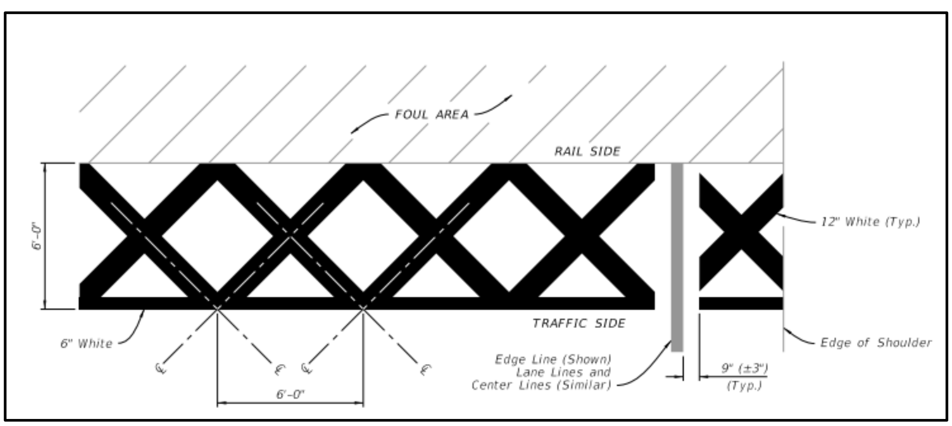
D.13 Railroad Dynamic Envelope Pavement Marking and Signage

Railroad Dynamic Envelope pavement markings should be used to delineate the area around at-grade railroad crossings where vehicles should not stop. The U. S. Department of Transportation's (U.S. DOT) Volpe Center found that the addition of the dynamic envelope pavement markings and modified signage reduced the number of vehicles that stopped within the dynamic envelope zone and increased the number of vehicles that stopped behind the stop line. The research was published as a presentation and called ***Evaluation of Pavement Markings within the Dynamic Envelope***. Coordination with the railroad is necessary. See ***Part 8 of the MUTCD*** for additional requirements for signage.

Where local roads cross state owned rail corridors, the railroad dynamic envelope pavement marking is required.

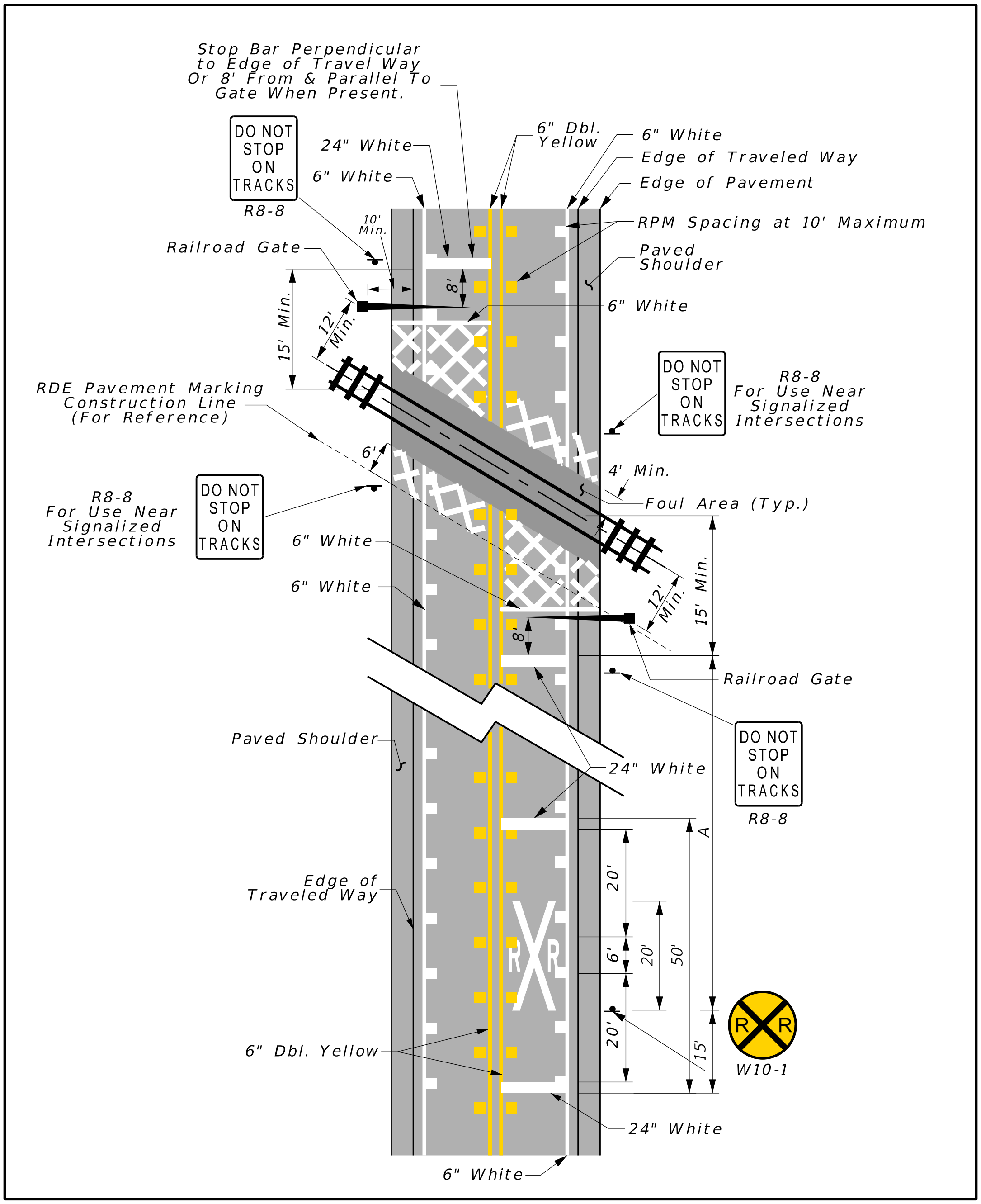
Figure~~s~~ 7 – 9 Railroad Dynamic Envelope Pavement Marking Detail, Figure 7 – 10 Railroad Crossing at 2-Lane Roadway, Figure 7 – 11 Railroad Crossing at Multilane Roadway, and Figure 7 – 12 Railroad Crossing at Multilane Roadway with Right Turn Lane provide examples of how rail dynamic envelopes can be signed and marked for at-grade rail crossings. Table 7 – 4 Location of "Do Not Stop on Tracks" Signage for Railroad Crossings Using the Rail Dynamic Envelope shows the distance between the RR Warning Sign (W10-1) and the Do Not Stop on Tracks (R8-8) sign.~~.~~ For additional information see the FDOT's~~e~~ ~~***FDOT's***~~ ***Standard Plans***.

Figure 7 – 9 Railroad Dynamic Envelope Pavement Marking Detail



Notes: 1. The pavement markings shall begin a minimum of 4' from the edge of the nearest rail or outside the foul area, as determined by the railroad owner.

Figure 7 – 10 Railroad Crossing at 2-Lane Roadway



Note: 1. For distance "A", see Table 7 – 4 Location of "Do Not Stop on Tracks" Signage for Railroad Crossings Using the Rail Dynamic Envelope.