(e max =0.10). These rates are based on Method 5 from the ***2011 AASHTO Greenbook*** using a maximum rate of 0.10 foot per foot of roadway width. Table 3 – 10 also provides the minimum radius required for normal crown without superelevation.

C.4.c.2 Low Speed Urban Roadways

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

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

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

Low Speed Urban Arterials and Collectors: Superelevation rates for low speed urban arterials and collectors are provided in Table 3 – 11 Superelevation Rates for Low Speed Arterials and Collectors (emax = 0.05). These rates are based on the Department's superelevation criteria for low speed arterials and collectors. Table 3 – 11 also provides the minimum radius required normal crown without superelevation.

Low Speed Local Roads: Minimum radii for design superelevation rates for low speed local roads are provided in Table 3 – 12 Minimum Radii (feet) for Design Superelevation Rates, Low Speed Local Roads (emax = 0.05). These rates are based on Method 2 from the 2011 AASHTO Greenbook. Table 3 – 12 also provides the minimum radius required for normal crown (-0.02 ft/ft) without superelevation.

Although superelevation is advantageous for traffic operations, various factors combine to make its use impractical in many built-up areas. Such factors include:

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

Table 3 – 10 Superelevation Rates for Rural Highways, Urban Freeways and High Speed Urban Highways (e max = 0.10)

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

* NC/RC and RC/e Break Points (Radius in feet)									
Break Points	Design Speed (mph)								
	30	35	40	45	50	55	60	65	70
R_{NC}	3349	4384	5560	6878	8337	9949	11709	13164	14714
R_{RC}	2471	3238	4110	5087	6171	7372	8686	9783	10955
e = NC if R ≥ R_{NC}					e = RC if R < R_{NC} and R ≥ R_{RC}				

NC = Normal Crown (-0.02) RC = Reverse Crown (+0.02)

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

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

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

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

NC = Normal Crown (-0.02) RC = Reverse Crown (+0.02)

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

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

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

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

C.4.d Maximum Curvature/Minimum Radius

Where a directional change in alignment is required, every effort should be made to utilize the smallest degree (largest radius) curvature possible. The use of the maximum degree of curvature should be avoided when possible. Design speed maximum degree of curvature or minimum radius for the maximum superelevation rates are provided in Tables 3 – 10 Superelevation Rates for Rural Highways, Urban Freeways and High Speed Urban Highways, 3 – 11 Superelevation Rates for Low Speed Arterials and Collectors, and 3 – 12 Minimum Radii (feet) for Design Superelevation Rates Low Speed Local Roads. The use of sharper curvature would call for superelevation beyond the limit considered practical or for operation with tire friction beyond safe or comfortable limits or both. The maximum degree of curvature or minimum radius is a significant value in alignment design.

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

Superelevation runoff is the general term denoting the length of street or highway needed to transition the change in cross slope from a section with the adverse crown removed (level) to the fully superelevated section, or vice versa. Tangent runoff is the general term denoting the length of street or highway needed to accomplish the change in cross slope from a normal cross section to a section with the adverse crown removed, or vice versa. Spiral curves can be used to transition from the tangent to the curve. Where the spiral curve is employed, its length is used to make the entire superelevation transition.

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

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

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

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

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

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

Table 3 – 13 Superelevation Transition Slope Rates

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

High Speed Roadways:

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

Low Speed Roadways:

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

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

C.4.f Sight Distance on Horizontal Curves

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

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

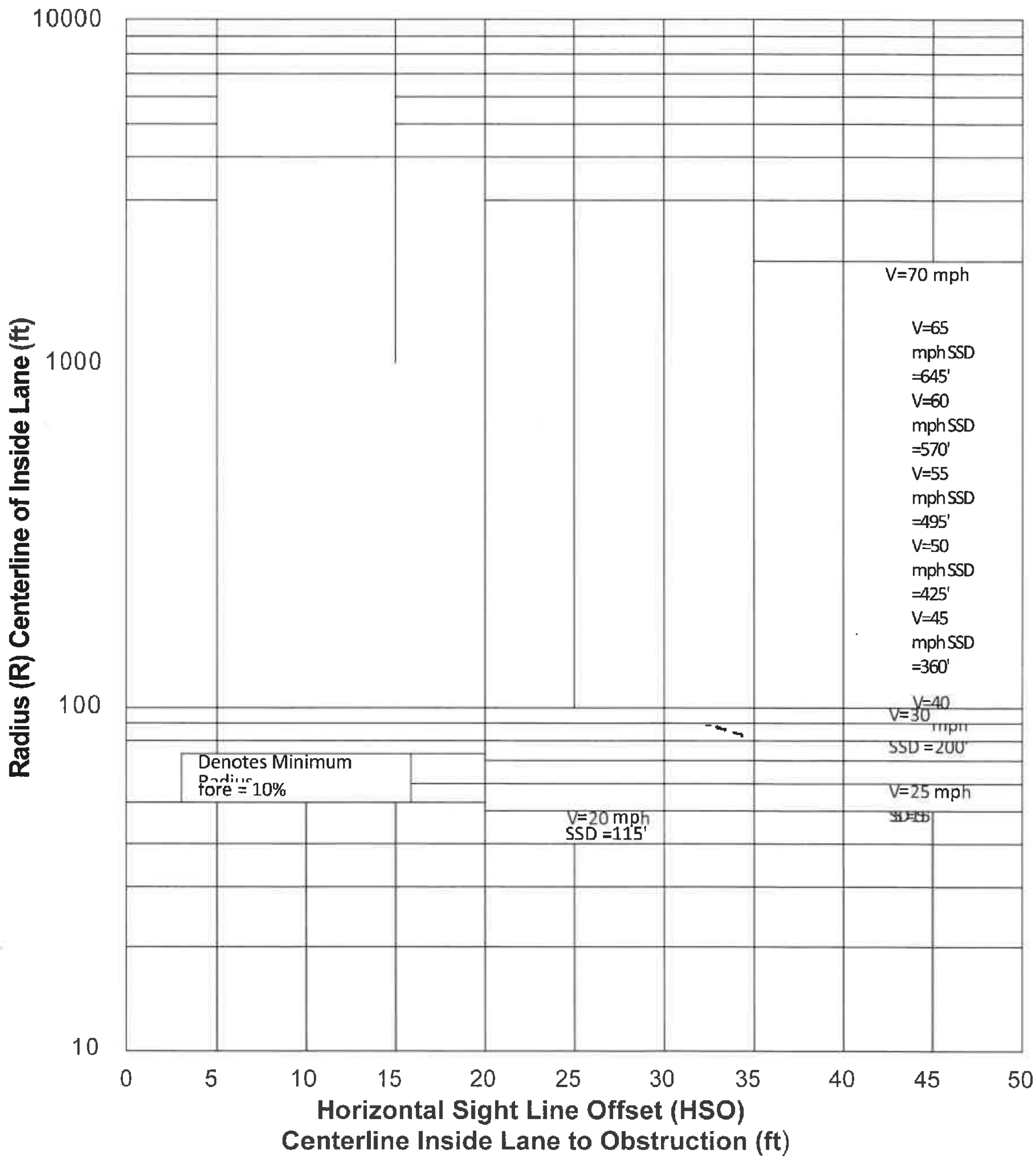
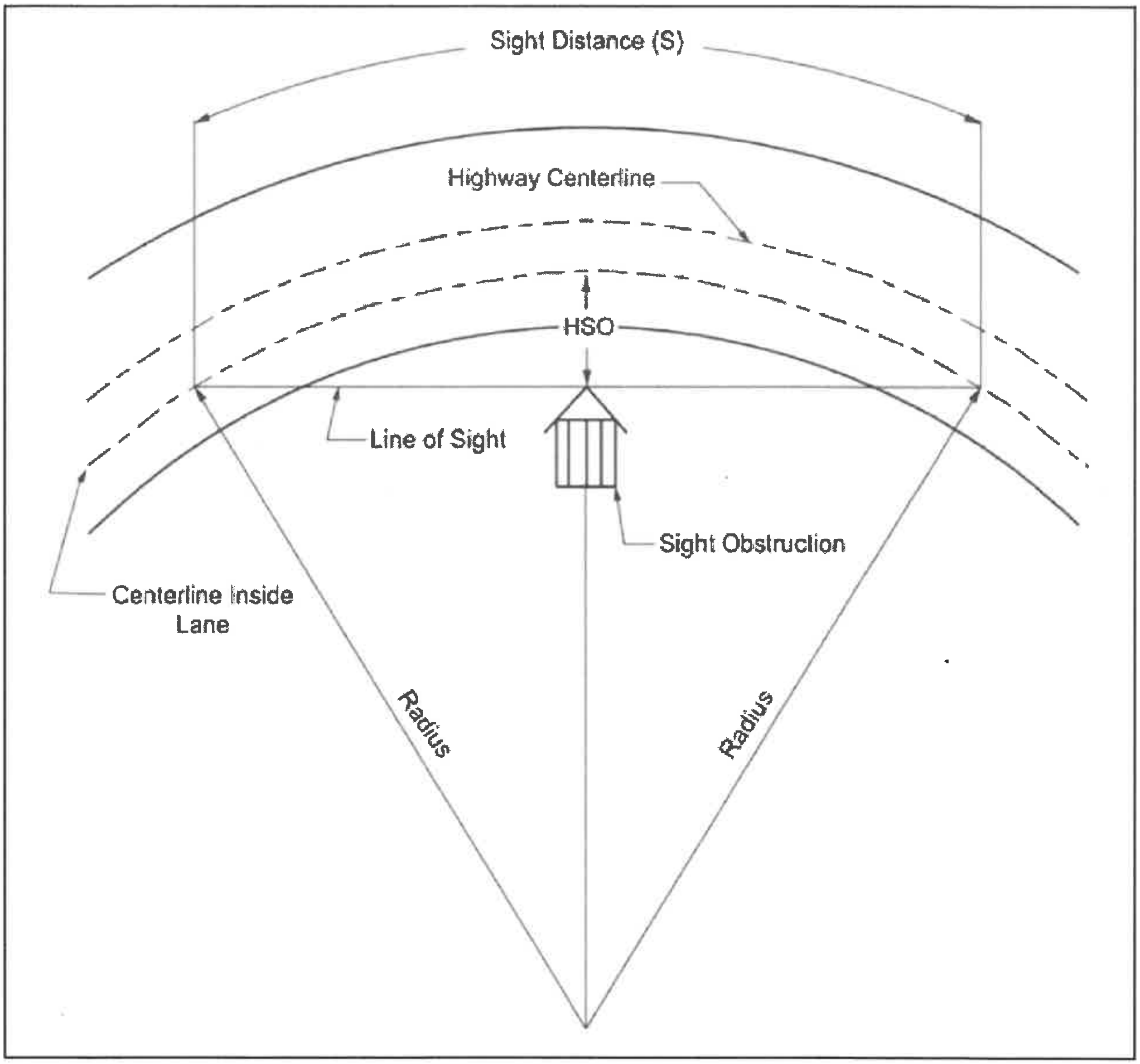


Figure 3 – 1B Diagram Illustrating Components for Determining Horizontal Sight Distance



HSO – Horizontal Sight Distance

Source: 2011 AASHTO Greenbook, Figure 3 – 23. Diagram Illustrating Components for Determining Horizontal Sight Distance

Table 3 – 14 Horizontal Curvature

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

C.4.g Lane Widening on Curves

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

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

Radius of Curve (feet)	Roadway width = 24 feet.							Roadway width = 22 feet.							Roadway width = 20 feet.						
	30	35	40	45	50	55	60	30	35	40	45	50	55	60	30	35	40	45	50	55	60
7000																					
6500																					
	12.0																				

Table 3 – 15B Adjustments for Traveled Way Widening Values on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

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

Source: 2011 AASHTO Greenbook, Table 3 - 27 Adjustments for Traveled Way Widening Values on Open Highway Curves.

Notes: 1. Adjustments are applied by adding to or subtracting from the values in Table 3-15A.

2. Adjustments depend only on radius and design vehicle; they are independent of traveled way width and design speed.

3. For 3-lane roadways, multiply above values by 1.5.

4. For 4-lane roadways, multiply above values by 2.0.

C.5 Vertical Alignment

C.5.a General Criteria

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

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

C.5.b Grades

The grades selected for vertical alignment should be as flat as practical, and should not be greater than the value given in Table 3 – 16 Maximum Grades in Percent.

For streets and highways requiring long upgrades, the maximum grade should be reduced so the speed reduction of slow-moving vehicles (e.g., trucks and buses) is not greater than 10 mph. The critical lengths of grade for these speed reductions are shown in Figure 3 – 2 Critical Length Versus Upgrade. Where reduction of grade is not practical, climbing lanes should be provided to meet these speed reduction limitations.

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

Recommended minimum gutter grades:

Rolling terrain - 0.5%   Flat terrain - 0.3%

Table 3 – 16 Maximum Grades (in Percent)

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

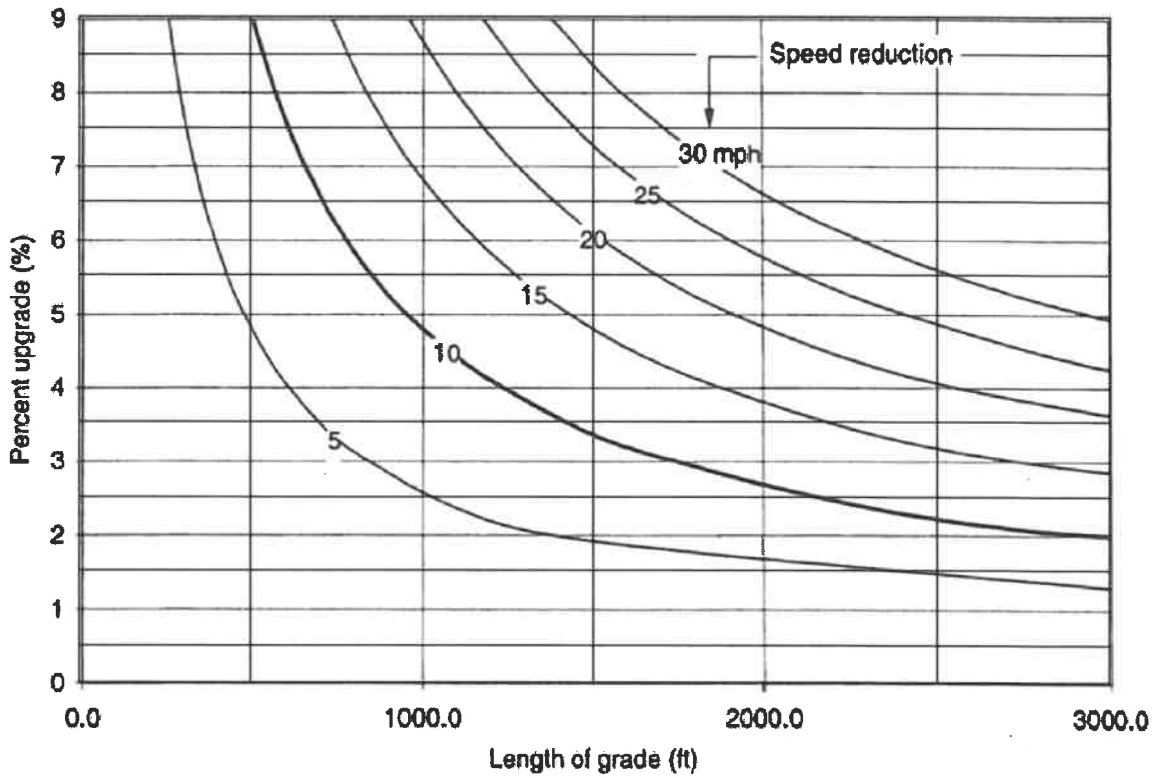
Source: 2011 AASHTO Greenbook, Tables 5–2, 6–2, 6–8, 7–2, 7–4, 8–1.

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

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

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

Figure 3 – 2 Critical Length Versus Upgrade



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

Source: 2011 AASHTO Greenbook, Figure 3-28.

C.5.c Vertical Curves

Changes in grade should be connected by a parabolic curve (the vertical offset being proportional to the square of the horizontal distance). Vertical curves are required when the algebraic difference of intersecting grades exceeds the values given in Table 3 – 17 Maximum Change in Grade Without Using Vertical Curve. Table 3 – 18 Rounded K Values for Minimum Lengths Vertical Curves provides additional information.

The length of vertical curves on a crest, as governed by stopping sight distance, is obtained from Figure 3 – 3 Length of Crest Vertical Curve (Stopping Sight Distance). The minimum length for passing sight distance on crest vertical curves shall be based on the K-values as shown in Table 3 – 19 Design Controls for Crest Vertical Curves (Passing Sight Distance). The minimum length of a sag vertical curve on open road conditions, as governed by vehicle headlight capabilities, is obtained from Figure 3 - 4 Length of Sag Vertical Curve (Headlight Sight Distance).

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

Table 3 – 17 Maximum Change in Grade Without Using Vertical Curve

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

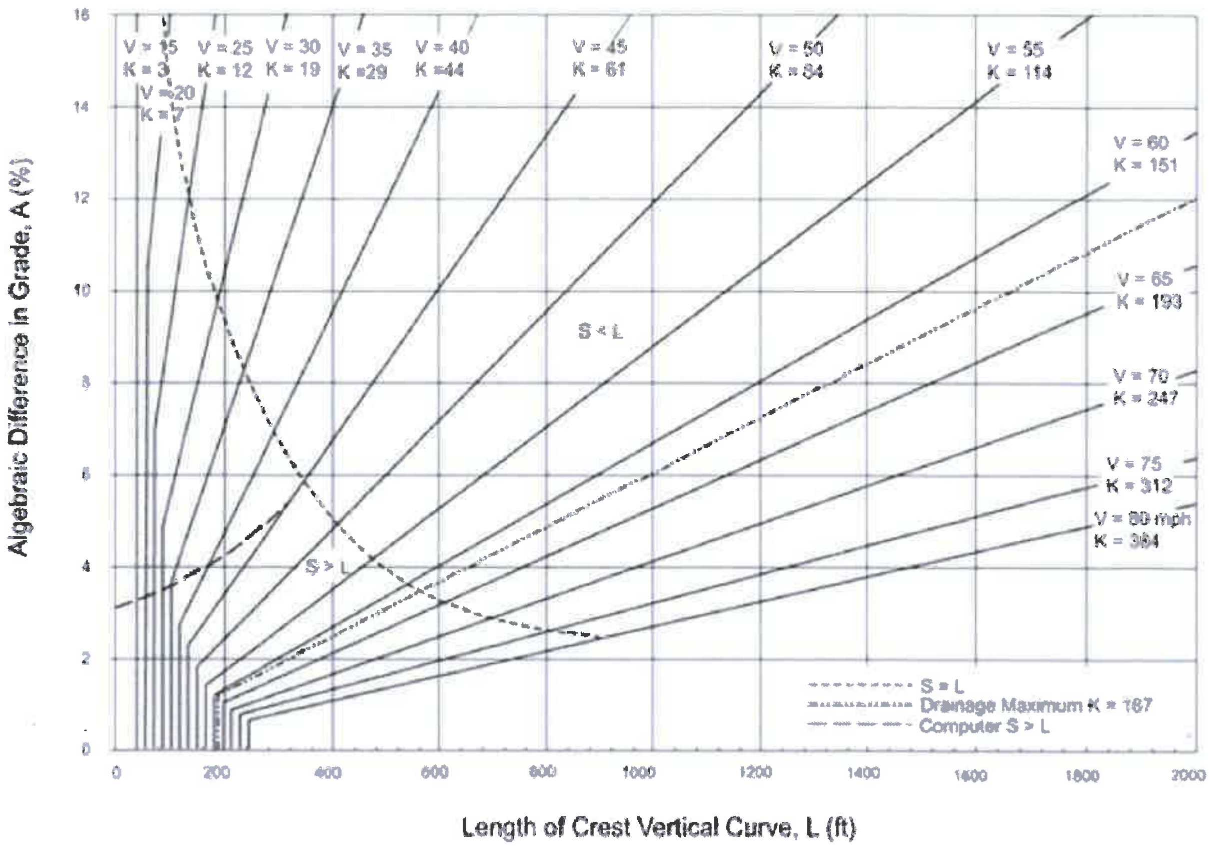
Table 3 – 18 Rounded K Values for Minimum Lengths Vertical Curves (Stopping Sight Distance)

(Based upon an eye height of 3.50 feet and an object height of 2 feet above the road surface)											
L = KA L = Length of Vertical Curve, A = Algebraic Difference of Grades in Percent											
Design Speed (mph)	20	25	30	35	40	45	50	55	60	65	70
K Values for Crest Vertical Curves	7	12	19	29	44	61	84	114	151	193	247
K Values for Sag Vertical Curves	17	26	37	49	64	79	96	115	136	157	181

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

Minimum Lengths for Vertical Curves on Collectors, Arterials, and Freeways (feet)			
Design Speed (mph)	50	60	70
Crest Vertical Curves (feet)	300	400	500
Sag Vertical Curves (feet)	200	300	400

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



Source: Figure 3-43 Design Controls for Crest Vertical Curves – Open Road Conditions, 2011 AASHTO Greenbook

Lengths of crest vertical curves are computed from the formulas:

When S is less than L, $L=AS^2/_2158$

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

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

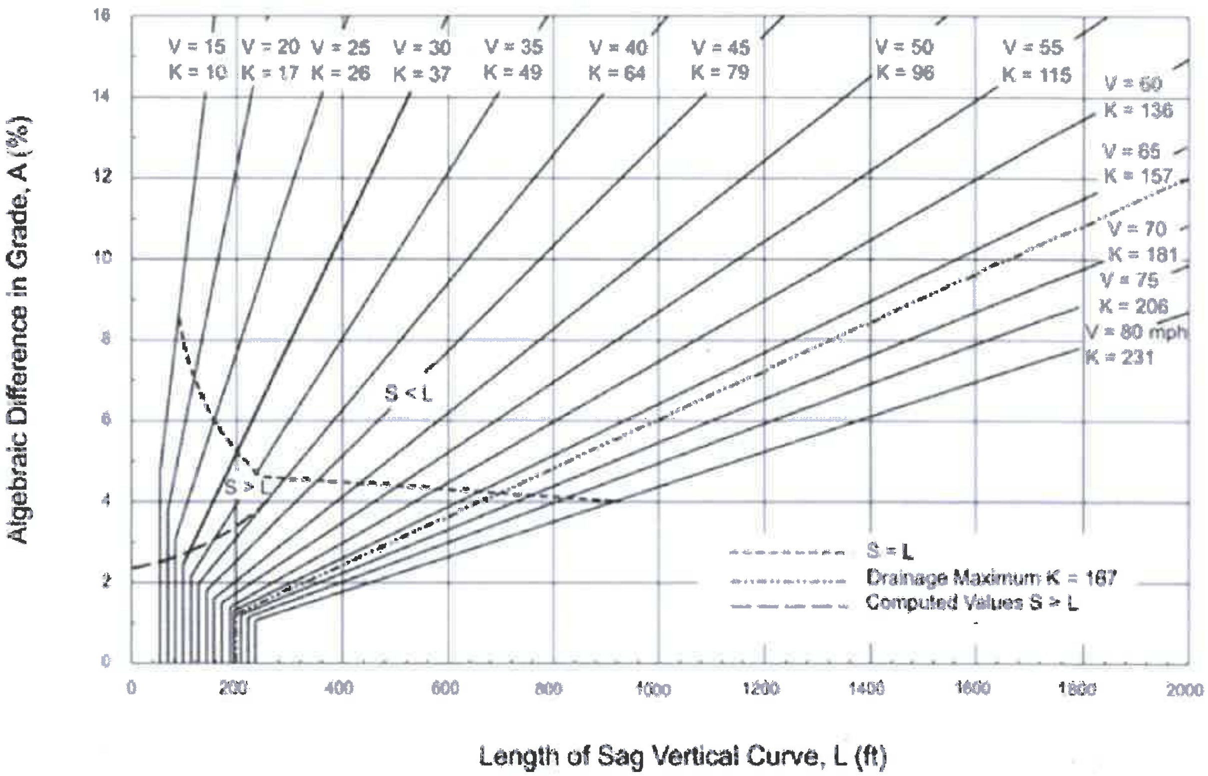
Table 3 - 19 Design Controls for Crest Vertical Curves (Passing Sight Distance)

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

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

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

Figure 3 – 4 Length of Sag Vertical Curve (Open Road Conditions)



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

Lengths of sag vertical curves are computed from the formulas:

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

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

L = Length of Sag Vertical Curve, feet
A = Algebraic Difference in Grades, percent
S = Light Beam Distance, feet

C.6 Alignment Coordination

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

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

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

important. Sight distances above the minimum are desirable at these locations.

- Alignment should be designed to enhance scenic views for the motorists.
- In residential areas, the alignment should be designed to minimize nuisance to the neighborhood.

C.7 Cross Section Elements

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

C.7.a Number of Lanes

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

C.7.b Pavement

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

C.7.b.1 Pavement Width

Minimum lane widths for travel lanes, speed change lanes, turn lanes and passing lanes are provided in Table 3 – 20 Minimum Lane Widths. The table applies to both divided and undivided facilities. For Information on parking lanes, see Section C.7.h Parking of this Chapter.

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

Table 3 – 20 Minimum Lane Widths

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

See Footnotes on next page

Footnotes

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

C.7.b.2 Traveled Way Cross Slope (not in superelevation)

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

C.7.c Shoulders

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

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

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

C.7.c.1 Shoulder Width

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

The minimum width of outside and median shoulders is provided in Table 3 – 21 Minimum Shoulder Widths for Flush Shoulder Highways. Shoulders for two-lane, two-way highways are based upon traffic volumes. Shoulder widths for multi-lane highways are based upon the number of travel lanes in each direction. Where bicyclists or pedestrians are to be accommodated on the shoulder, a minimum usable width of 4 feet is required (5 feet if adjacent to a barrier). On approaches to narrow bridges where the paved shoulder is reduced, the Department's ***Standard Plans Index 700-106*** provides information on signing and marking the approaching shoulder.

Table 3 – 21 Minimum Shoulder Widths for Flush Shoulder Highways

Two Lane Undivided

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

Multilane Divided

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

C.7.c.2 Shoulder Cross Slope

The shoulder serves as a continuation of the drainage system, therefore, the shoulder cross slope should be somewhat greater than the adjacent traffic lane. The cross slope of shoulders should be within the range given in Table 3 – 22 Shoulder Cross Slope.

Table 3 – 22 Shoulder Cross Slope

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

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

Source – 2011 AASHTO Greenbook, Section 4.4.3 Shoulder Cross Sections.

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

C.7.d Sidewalks

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

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

The decision to construct a sidewalk in a rural area should be based on engineering judgment, after observation of existing pedestrian traffic and expectation of additional demand, should a sidewalk be made available.

Sidewalks should be constructed as defined in this Manual. ***Chapter 8 – Pedestrian Facilities, Chapter 10 – Maintenance and Resurfacing*** and ***Section C.10.a.3 – Sidewalks and Curb Ramps*** of this chapter provide additional detailed information. ***AASHTO's Guide for the Planning, Design and Operation of Pedestrian Facilities (2004)***, and ***Section 4.17.1 Sidewalks*** of ***AASHTO's Policy on Geometric Design of Highways and Streets (2011)*** provide additional information.

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

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

C.7.e Medians

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

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

Median separation is required on the following streets and highways:

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

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

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

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

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

C.7.e.1 Type of Median

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

C.7.e.2 Median Width

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

The minimum permitted widths of freeway medians are given in Table 3 – 23 Minimum Median Width. Where the expected traffic volume is heavy, the widths should be increased over these minimum values. Median barriers shall be used on freeways when these minimum values are not attainable.

The minimum permitted median widths for multi-lane rural highways are also given in Table 3 – 23Minimum Median Width . On urban streets, the median widths shall not be less than the values given in Table 3 – 23. Where median openings or access points are frequent, the median width should be increased.

The minimum median widths given in these Tables may have to be increased to meet the requirements for cross slopes, drainage, and turning movements (C.9 Intersection Design, this chapter). The median area should also include adequate additional width to allow for expected additions of through lanes and left turn lanes. Where the median width is sufficient to produce essentially two separate, independent roadways, the left side of each roadway shall meet the requirements for roadside clear zone. Changes in the median width should be accomplished by gently flowing horizontal alignment of one or both of the separate roadways.

Table 3 – 23 Minimum Median Width

Type of Facility	Width (feet)

Freeways	
Freeways, Without Barrier	---
Design Speed ≥ 60 mph	60
Design Speed < 60 mph	40
All, With Barrier, All Design Speeds	26 [1]
Arterial and Collectors	
Design Speed ≥ 50 mph	40
Design Speed ≤ 45 mph	22 [2]
Paved and Painted for Left Turns	See Table 3 – 17 Minimum Lane Widths

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

Footnotes:

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

C.7.e.3 Median Slopes

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

The median cross slope should not be steeper than1:6 (preferably not steeper than 1:10). The depth of depressed medians may be controlled by drainage requirements. Increasing the width of the median, rather than increasing the cross slope, is the proper method for developing the required median depth.

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

C.7.e.4 Median Barriers

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

C.7.f Islands

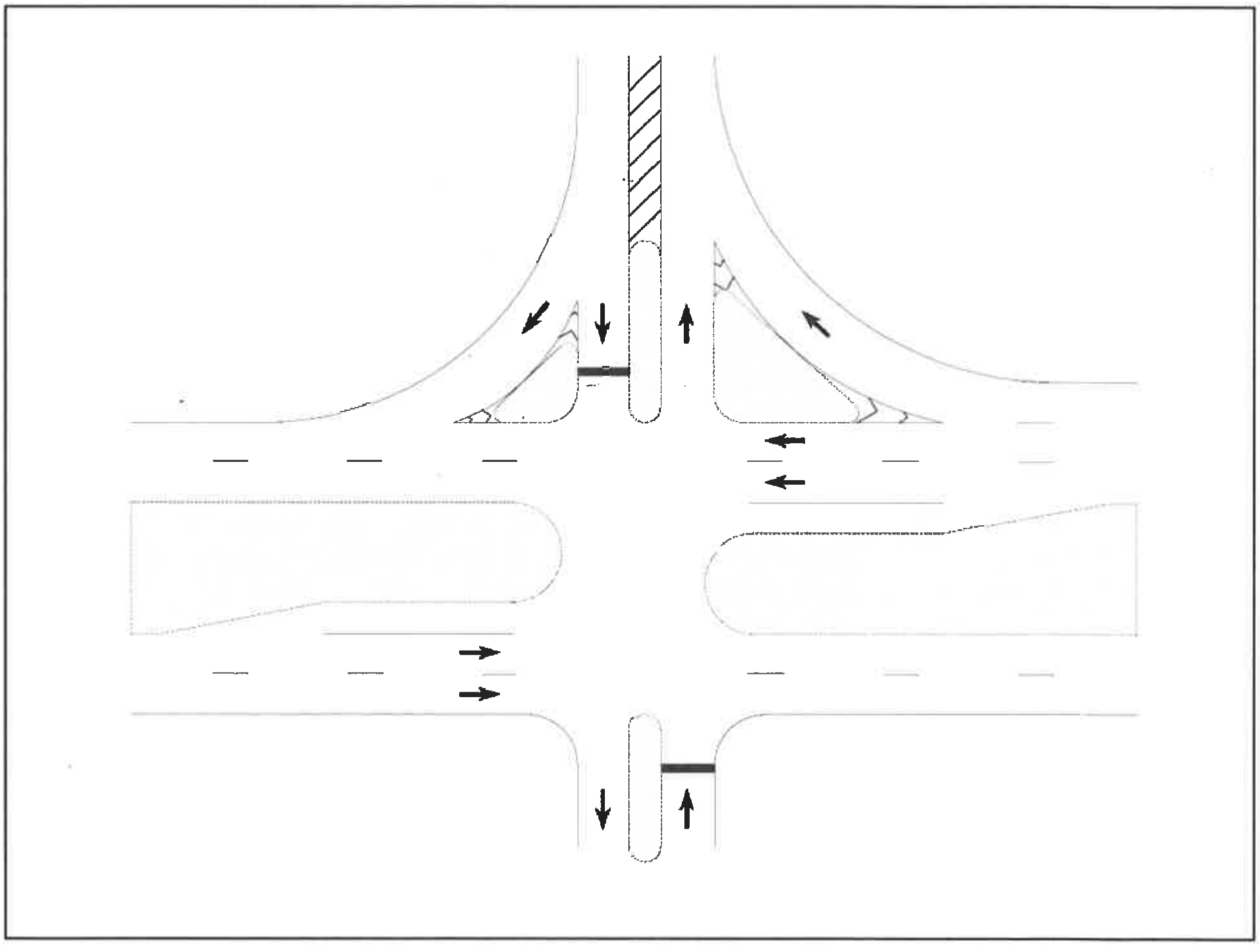
An island is a defined area between traffic lanes used for control of vehicle movements. Most islands combine two or more of these primary functions:

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

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

The dimensions and details depend on the intersection design as illustrated in Figure 3 – 5 General Types and Shapes of Islands and Medians. They should conform to the general principles that follow.

Figure 3 – 5 General Types and Shapes of Islands and Medians



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

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

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

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

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

Careful consideration should be given to the location and type of plantings. Plantings, particularly in narrow islands, may create problems for maintenance activities. Plantings and other landscaping features in channelization areas may constitute roadside obstacles and should be consistent with the ***AASHTO Roadside Design Guide.***

C.7.f.1 Channelizing Islands

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

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

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

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

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

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

Figure 3 – 6 Channelization Island for Pedestrian Crossings (Curbed)

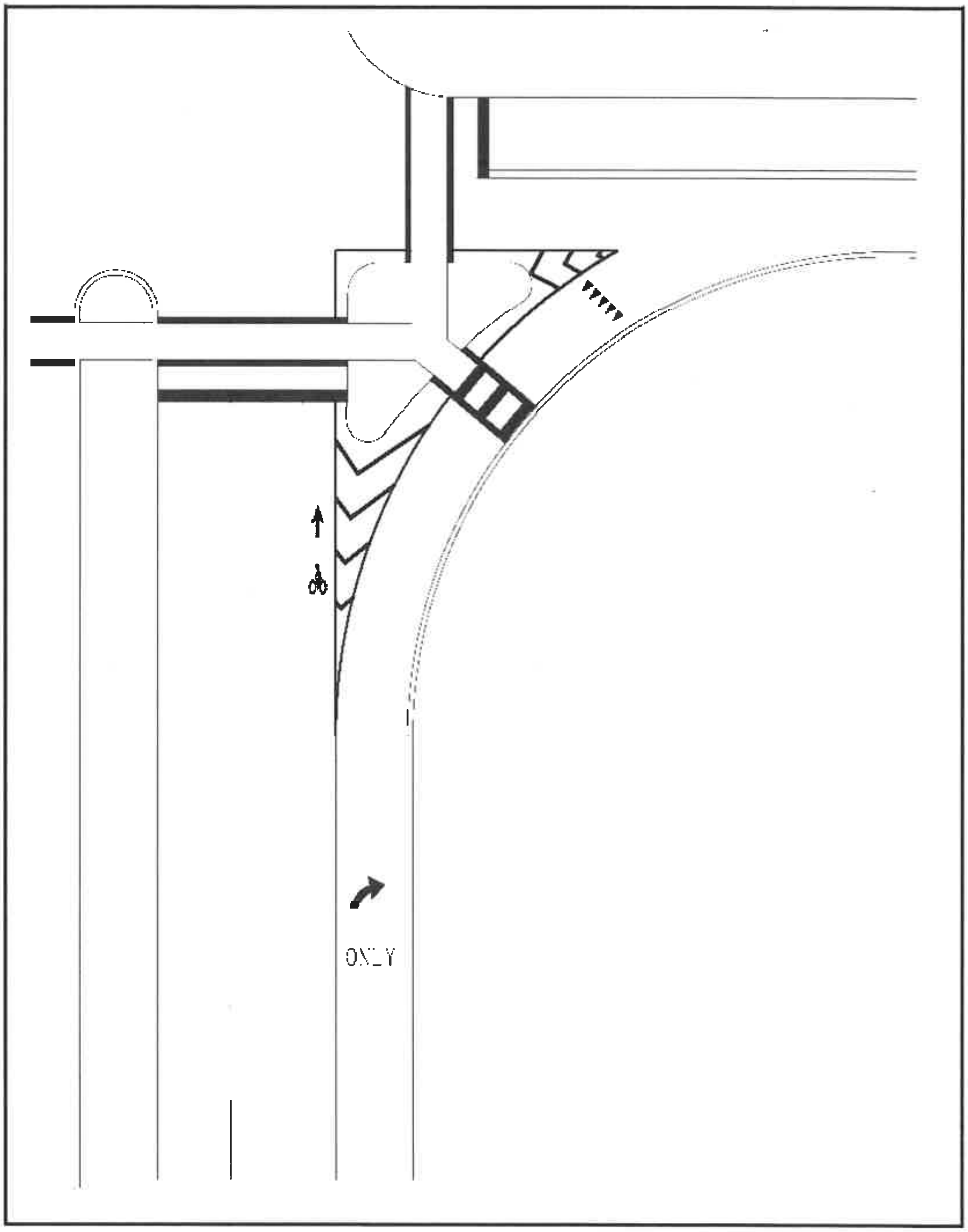
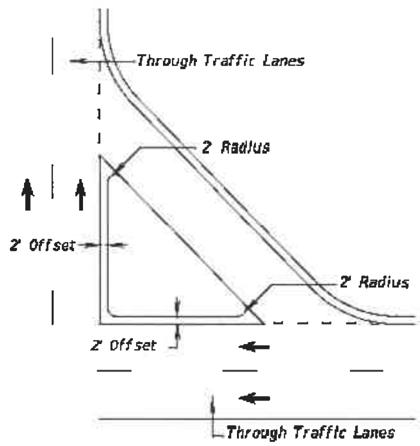
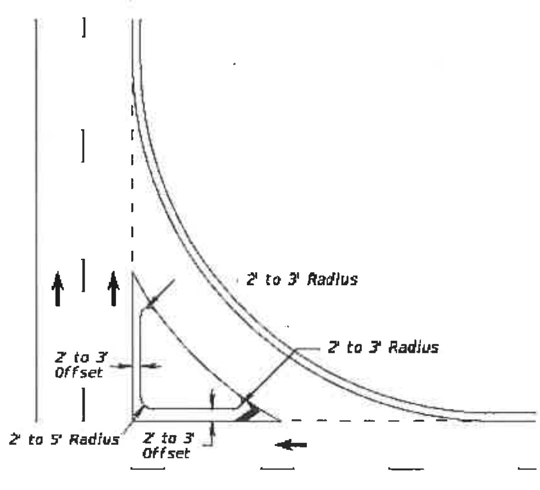


Figure 3 – 7 Details of Corner Island for Turning Roadways (Curbed)

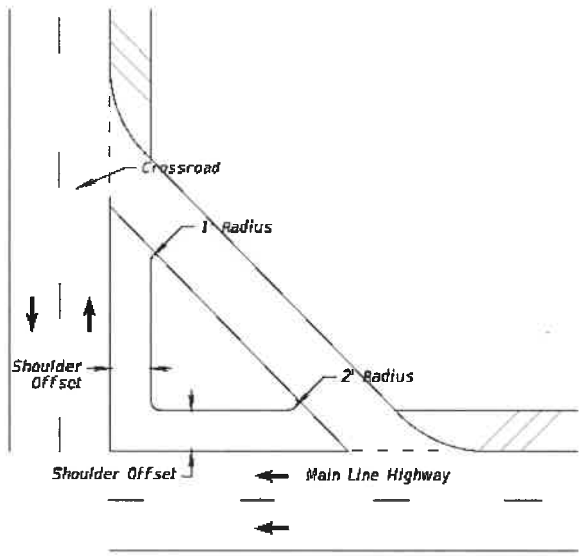


Small

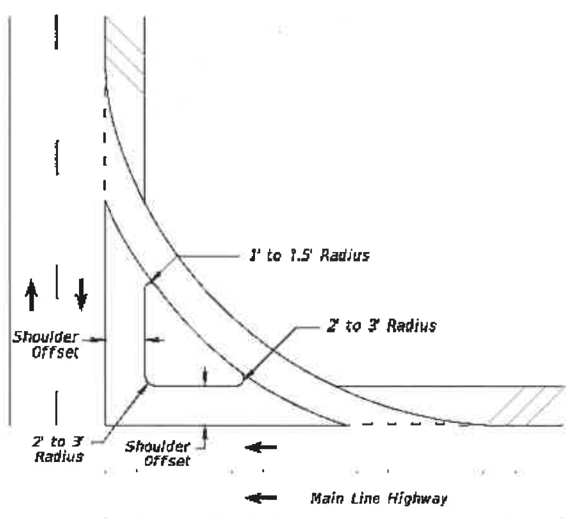


Intermediate

Figure 3 – 8 Details of Corner Island for Turning Roadways (Flush Shoulder)



Small



Intermediate / Large

C.7.f.2 Divisional Islands

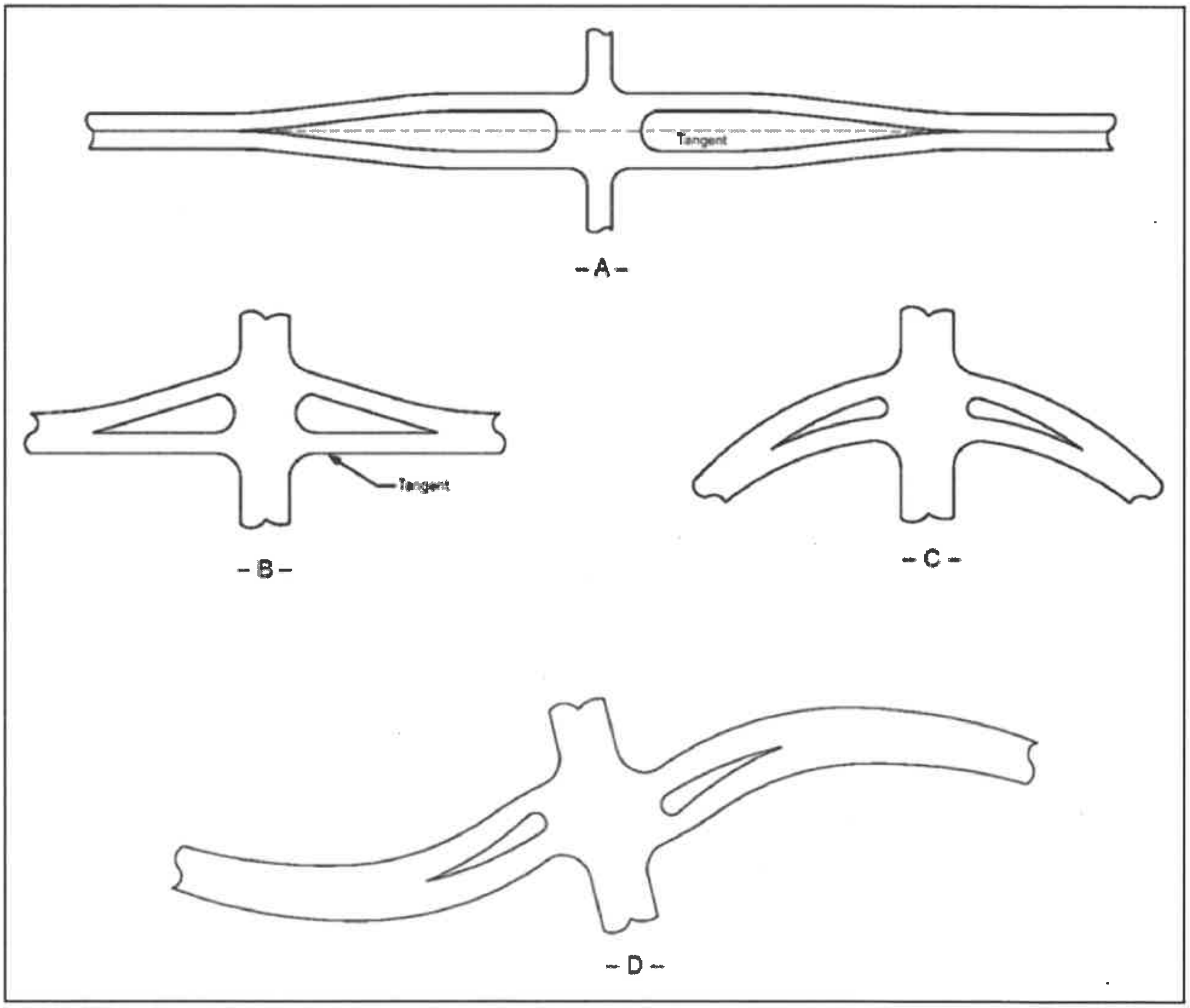
Divisional islands often are introduced on undivided highways at intersections. They alert drivers to the crossroad ahead and regulate traffic through the intersection. These islands are particularly advantageous in controlling left turns at skewed intersections and at locations where separate roadways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

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

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

Where an island is introduced at an intersection to separate opposing traffic on a four-lane road or on a major two-lane highway carrying high volumes, two full lanes should be provided on each side of the dividing island (particularly where future conversion to a wider highway is likely). In other instances, narrower roadways may be used. For moderate volumes, roadway widths shown under Case II (one-lane, one-way operation with provision for passing a stalled vehicle) in Table 3 - 34 Derived Pavement Widths for Turning Roadways for Different Design Vehicles are appropriate. For light volumes and where small islands are needed, widths on each side of the island corresponding to Case I in Table 3 – 34 may be used

Figure 3 – 9 Alignment for Divisional Islands at Intersections



C.7.f.3 Refuge Islands

A refuge island for pedestrians at or near a crosswalk or shared use path crossing aids pedestrians and bicyclists who cross the roadway. Raised-curb corner islands and center channelizing or divisional islands can be used as refuge areas. Refuge islands for pedestrians and bicyclists crossing a wide street, for loading or unloading transit riders, or for wheelchair ramps are used primarily in urban areas. Figure 3 – 10 Pedestrian Refuge Island, Figure 3 – 11 Pedestrian

Crossing with Refuge Island (Yield Condition), and Figure 3 – 12 Pedestrian Crossing with Refuge Island (Stop Condition) show divisional islands that support a midblock crosswalk with stop and yield conditions. The distance A shown in the figures is based upon the ***MUTCD***, and shown following the figures.

The location and width of crosswalks, the location and size of transit loading zones, and the provision of curb ramps influence the size and location of refuge islands. Refuge islands should be a minimum of 6 feet wide. Pedestrians and bicyclists should have a clear path through the island and should not be obstructed by poles, sign posts, utility boxes, etc. Sidewalk and shared use path curb ramps in islands shall meet the requirements found in ***Section C.10.a.4*** of this chapter and ***Chapter 8 – Pedestrian Facilities.*** Curb ramps that are part of a shared use path shall also meet the requirements of ***Chapter 9 – Bicycle Facilities.***

Figure 3 – 10 Pedestrian Refuge Island

North Main Street, Gainesville, FL

Figure 3 – 11 Pedestrian Crossing with Refuge Island (Yield Condition)

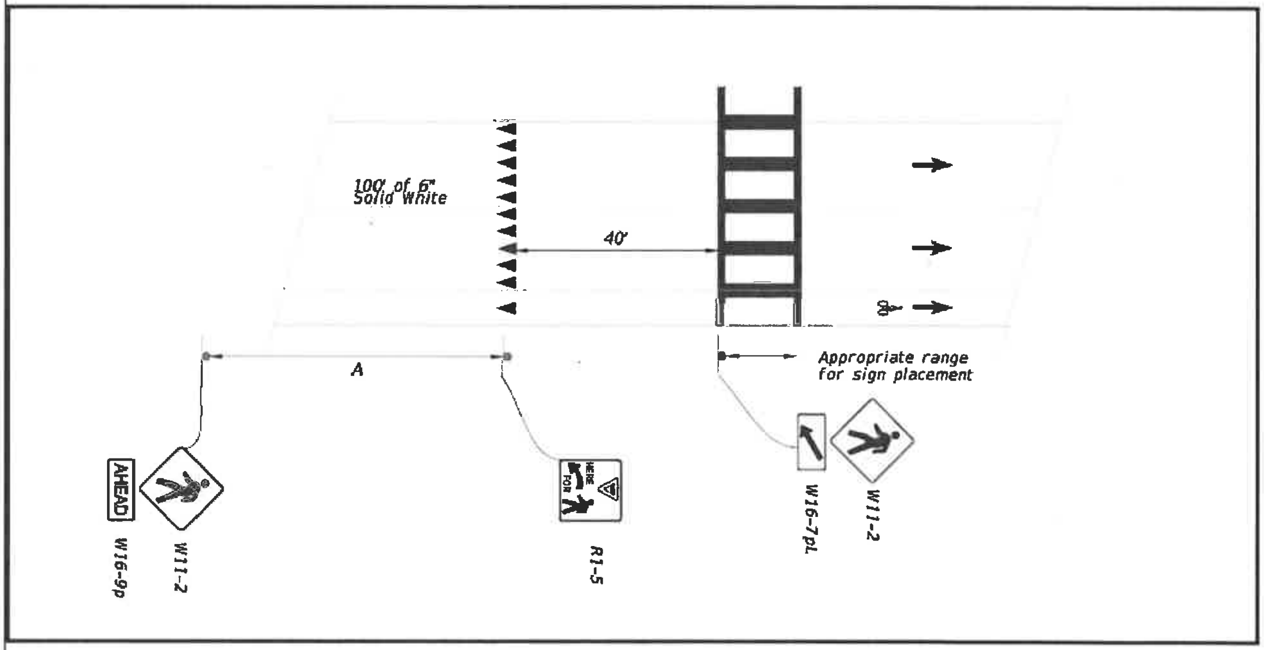
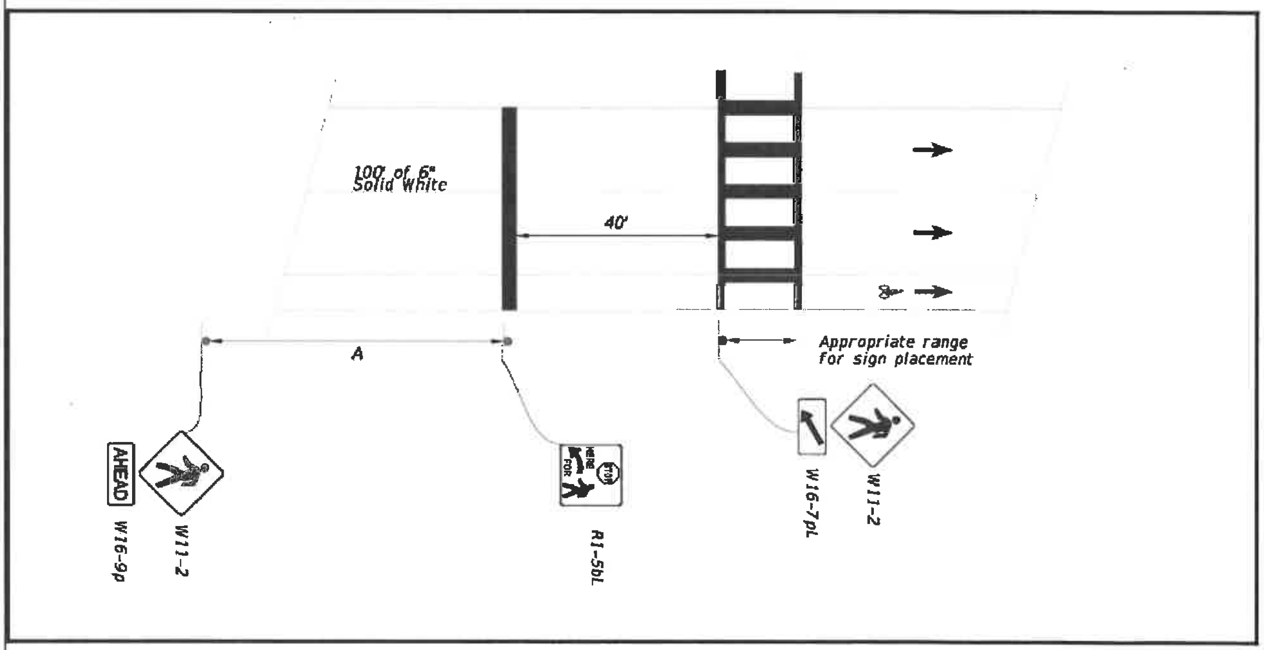


Figure 3 – 12 Pedestrian Crossing with Refuge Island (Stop Condition)



Note: 1. See following page for distance A.

The distance A shown in Figures 3 – 11 and 3 – 12 for the advance warning sign should be:

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

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

An example of a pedestrian crossing through a refuge island is shown in Figure 3 – 13 Pedestrian Crossing in Refuge Island. Other options are shown in the Department's ***Standard Plans 522-002 Detectable Warnings and Sidewalk Curb Ramps***.

Figure 3 – 13 Pedestrian Crossing in Refuge Island